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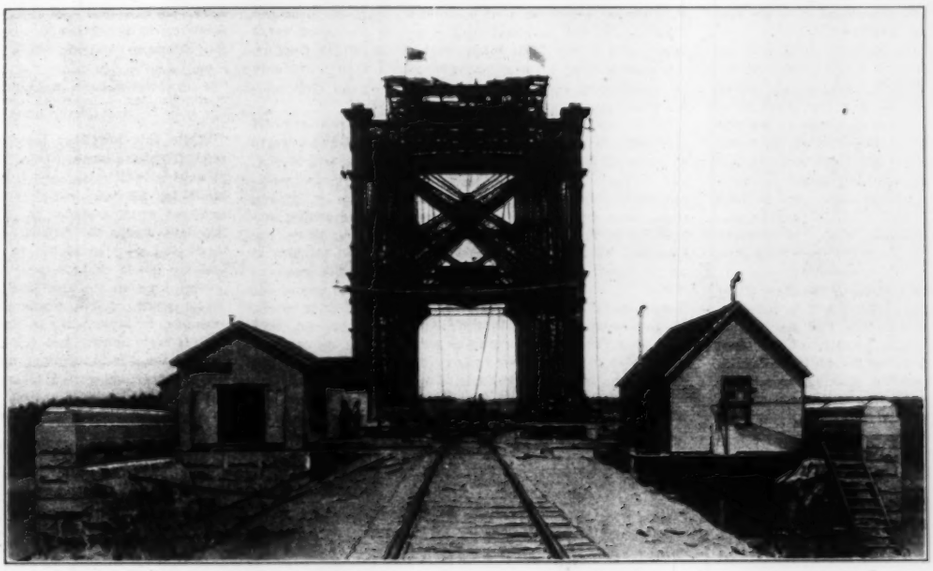
PROGRESS IN THE ERECTION OF THE QUEBEC BRIDGE.

Work on the erection of the great cantilever bridge across the St. Lawrence River at Quebec, which, with its main span of 1,800 ft., is to be the longest bridge span in the world, was resumed on April 17. The accompanying photographs, for which we are indebted to Mr. John Sterling Deans, M. Am. Soc. C. E., Chief Engineer of the Phoenix Bridge Co., contractors for the superstructure, show the completed portal and the shore arm of one of the main cantilevers. The third view shows the hoisting of two huge panels of eye-bars for the top chord.

We are informed that since the view of the shore arm was taken, the main center post of the cantilever has been placed and work has now been started on the erection of the channel arm of the main cantilever.

The false-work used for the erection of the shore arm at the east end, shown in the photograph, is being taken down and will be used for the erection of the shore arm at the west end.

A description of the structure was given in our issue of Sept. 14 last.



VIEW OF PORTAL OF ST. LAWRENCE RIVER BRIDGE. VIEW TAKEN MAY 19, 1906.

THE DEVELOPMENT OF WATER SUPPLIES AND WATER-SUPPLY ENGINEERING.*

By Frederic P. Stearns, President, Am. Soc. C. E.

It would be futile in a short address to attempt to trace the beginning of water-supply engineering. That it was not in its infancy in the Roman era is a matter of common knowledge, both from the carefully written descriptions of the water-works of that era and from the remains now to be seen of the magnificent arches and other works near Rome and in former Roman provinces, which prove the great ability of the water-works engineers of that period.

Of what may be called modern water-works, London probably furnishes the earliest example. Before these were built there were, as is often the case, minor works for bringing water from springs near the city. One of them was completed in the year 1285, and others from time to time afterward, and there was obtained at a still later date a larger supply by pumping water with tidal power from the Thames at London Bridge.

*Condensed from the Presidential Address before the Annual Convention of the American Society of Civil Engineers.

The first extensive modern water supply, however, was that provided by the New River Water-Works of London, built by Sir Hugh Middleton, in the four years from 1609 to 1613. The water was obtained from springs near the River Lea, distant about 20 miles in a direct line from London, but the open canal through which the water flowed to the city made many windings to avoid hills and valleys, so that it had a total length of 38 miles. The canal had an inclination of 3 ins. per mile; its width averaged 18 ft., and its depth seldom exceeded 5 ft. The springs failed to furnish the required quantity of water, and were supplemented by a direct connection with the river. These were very great works for those days, when, to quote words written in 1835, "the science and practice of civil engineering were very little known."

In 1633 London is said to have been well supplied with water from pipes in all streets, and in nearly every house the rent of which was from $75 to $100 per year.

After London was well supplied with water, 163 years elapsed before a public water supply worthy of notice was provided for any of the larger cities of America. The first one to be supplied was Boston, into which water was introduced by a water company in 1796. The water was taken by gravity from Jamaica Pond, situated a short distance outside the city limits. Philadelphia built its own works, which were supplied with water by pumping from the Schuylkill River, and were first operated in 1801. New York was first furnished with a water supply by the Manhattan Company at about the same time. In all these cities the water was distributed through hollow logs, which are sometimes found well preserved at the present day.

The water supplies of New York and Boston continued to be wholly inadequate to the needs of the cities for many years, as money was not plenty, and the cities were loath to embark upon great enterprises with which they were unfamiliar. After long delay, however, they proceeded to build permanent works on a most liberal scale. New York, especially, displayed great courage in deciding to build an aqueduct 40 miles long, having a capacity of 80,000,000 gallons per day, from the Croton River to the city. This work involved the great difficulty of conveying the water over the Harlem River at a height of more than 100 ft. above its surface.

Having once begun the construction of modern water-works, the rapid growth of American cities and the lavish use and waste of water have required a very rapid development of water-works systems in all parts of the country, and works of unprecedented magnitude are now being undertaken with less hesitation than the much smaller works of many years ago. The most notable of these recent undertakings is that of the City of New York for bringing water a distance of 100 miles from the Catskill Mountains, at an estimated cost of $162,000,000.

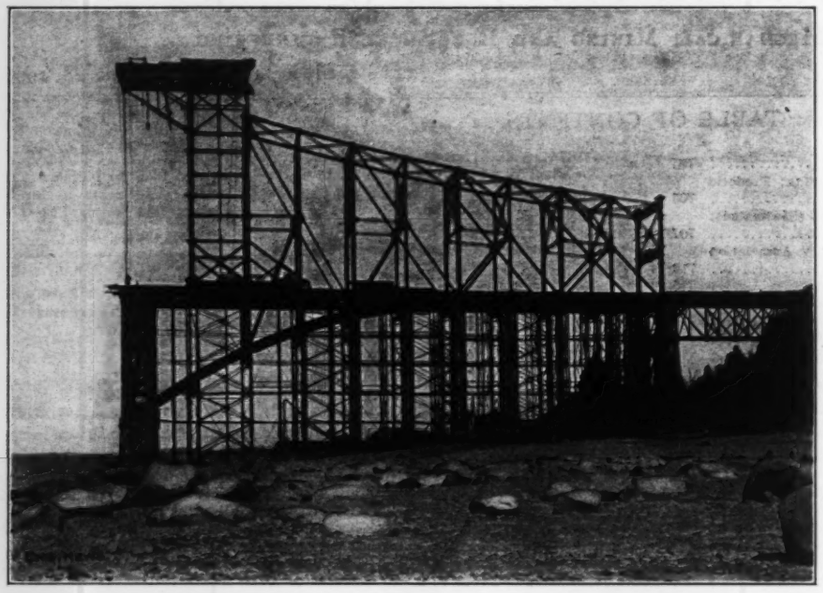
Let us now leave the development of water-works as a whole, and consider the development of their various parts and the influence which water-works engineering has had upon engineering along other lines.

The Romans were masters of the art of building masonry aqueduct bridges and aqueducts, and the advance upon their practice up to the present time is not very striking. The Romans excavated long tunnels—one of them is said to have had a length of 3 miles—through which to convey water, but these must have been especially difficult to excavate before the days of explosives. Their greatest trouble was that due to the absence of suitable pipes for conveying the water across low lands, and it was this difficulty which caused the construction of the aqueduct bridges. They had lead pipes, made by folding sheet lead and soldering the edges, and used tile and stone pipes in some instances, but none of them compared in efficiency with the modern iron pipe.

The date when iron pipes were first used for water-works is uncertain; there are instances of their use as far back as the middle of the eighteenth century, but they did not come into general use until after the beginning of the nineteenth century. In 1746 the Chelsea Water Co., of London, laid a 12-in. flanged pipe to a reservoir at Hyde Park, which was taken up and relaid in 1791 "in consequence of its joints being perished." Another pipe laid a few years later in the Kensington Gardens was taken up in 1819 on account of incrustations on the interior surface of the pipe. A 5-in. iron pipe was laid to supply water to Edinburgh in 1787. The London companies changed from wooden pipes, which were in general use, to cast-iron pipes, between the years 1810 and 1820.

Mr. Thomas Simpson, the engineer of the Chelsea Water-Works, of London, designed the first bell and spigot pipes, about the year 1785, and at that time laid a few lengths of this pipe with lead joints as an experiment.

Philadelphia in 1804 laid about three-fourths of a mile of iron pipe as an experiment, and in 1817 began to use



SOUTH ANCHOR ARM OF ST. LAWRENCE RIVER BRIDGE. VIEW TAKEN MAY 7, 1906.

cast-iron pipes with bells, similar to those now in use. These pipes were imported from England.

It is of interest to note that as early as 1810 a 15-in. flexible cast-iron main with ball and socket joints at intervals was laid across the River Clyde to supply water to a portion of Glasgow. A trench was first dredged in the bed of the river, then the pipes, attached to wooden frames, were connected on land on one side of the river, and subsequently dragged across it until one end reached the opposite shore, the operation being aided by pontoons, in much the same manner that such work is done at the present time. The credit for the design of this siphon is due to the celebrated James Watt, the inventor of the steam engine, who, for some years, practiced civil engineering.

"A patent Steam Engine" was erected by the New River Water Company, of London, in 1787, and it is possible, though not probable, that this was the first application of steam for pumping a city water supply. The steam engines first used for pumping water for Philadelphia, in 1801, were unique machines, as the walking-beams, flywheels, connecting-rods, and even the cold-water pumps, were made of wood. As indicating the difficulty of constructing such machines in this country at that time, it is said that the main steam cylinder of one of the engines was cast in two parts, and that nearly four months were occupied in boring it out fit for use. The boilers were wooden boxes bolted and braced on the outside. "The fire-box inside the boiler was of wrought iron, with vertical flues of cast iron."

In the art of dam building the engineers of water supply and of irrigation have been rivals for many years. In India, and in recent years in the arid lands of our own country, the irrigation engineers have been designing and constructing great dams of masonry and of earth. Up to the present time, however, the water-supply engineers have created the greater works. The New Croton Dam, completed within the current year, with a height from the lowest point of its foundation to its top of 297 ft., is the greatest of masonry dams, although the mass of masonry is small in comparison with that of the Great Pyramid. The San Leandro Dam of the Contra Costa Water Company, which supplies Oakland and other cities near San Francisco with water, is the highest of existing earth dams. It is an interesting feature that this dam, as well as three notable dams connected with the San Francisco Water-Works—two of earth and one of concrete—were shaken by the earthquake of April 18, 1906, when under full water pressure, without any serious result.

Although the art of dam building has made such progress that great dams like those just mentioned have been designed and built of sufficient strength to withstand safely the tests of practical use, the theory of the design of both masonry and earth dams is only imperfectly understood. The rules given in the text-books for designing masonry dams do not take account of some important forces affecting their stability, most of which are the result of temperature changes. Several of the more recent failures of masonry dams, most of them on Southern rivers, could without doubt be traced to the neglect of the effect of forces which are not sufficiently considered in making designs.

Experiments made during the last twenty years to determine the laws governing the filtration of water through sand and soils of various kinds have furnished much additional information for use in designing earth dams, and it has been found that, with proper designs, earth containing a large percentage of fine particles may be substituted successfully for a masonry or clay puddle corewall.

There are some water-supply features which are not creditable to the American people. First, the great waste of water which prevails in most of our cities, and second, the taking of water directly from polluted rivers without either filtration or long storage. There is, among those who govern our large cities, an unreasoning opposition to the introduction of water meters, which are conceded by nearly all who know about such matters to be the most efficient means for diminishing the waste of water. The recent action by the government of one of our larger cities emphasizes this point. This attitude seems to have some relation to the commonly expressed view that water should be as free as air, and that on sanitary grounds it is not well to restrict its use, but no one has yet demonstrated the sanitary advantages of a leaky faucet or a defective ball-cock. Experience shows that meters prevent waste, but do not prevent the use of as much water as is required for sanitation. Waste not only adds greatly to the difficulty and expense of [illegible] sufficient water supply, and requires the ex[illegible] money which might well be applied to im[illegible] quality of the water by filtration or otherwise. [illegible] greatly to the difficulty and expense of the s[illegible] tem, especially where pumping and purifica[illegible] sewage are necessary. There is undoubtedly [illegible] growing opinion among water-works engineers [illegible] connected with water-works management th[illegible] eral application of water meters is desirable [illegible] public and the governing bodies of our cities [illegible] become educated so that they will accept th[illegible]

While I have said that the taking of wa[illegible] from polluted rivers is not creditable to th[illegible] people, I should add that under the leader[illegible] engineers and sanitarians of the present da[illegible] creditable feature is fast disappearing. There [illegible] be said, as an excuse for the practice years [illegible] many such water systems were inaugurated [illegible] purity of water was then judged mainly from [illegible] standpoint. Bacteriology was unknown, and [illegible] of filtration was not appreciated. To show t[illegible] view held thirty-three years ago, I quote fro[illegible] written by one of the most prominent water-w[illegible] neers of that time:

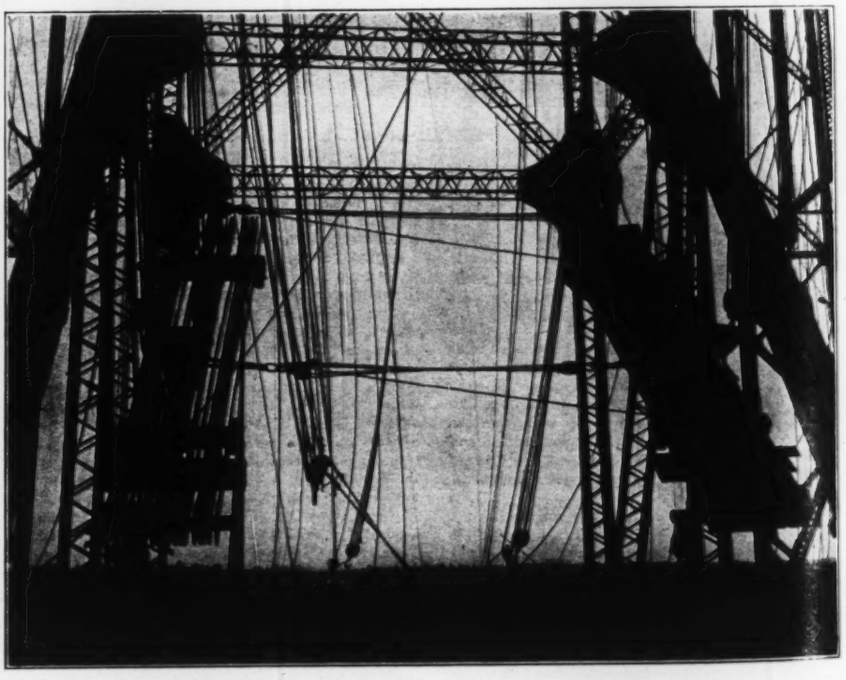
The object of filtration is to remove the [illegible] purities of water and render it colorless. . . . [illegible] also, to a limited extent, a lessening of the ch[illegible] purity.

On the other hand, although the effect of fi[illegible] rendering water wholesome was not so well known, there was ample precedent for filtering river waters, as it had been practiced in Great Britain since about the year 1811. In 1827 there were enough places where filter-beds were in use in England and Scotland to induce Mr. Thomas Simpson to make a trip of investigation, and he then visited filtration works in various places which were said to have proved completely efficient. In 1835, a writer, after referring to the entire success of the filter-beds built by Mr. Simpson in 1829, offers this prophecy:

Probably at no very distant period the desirable practice of filtering the whole of the water supplied from rivers to the inhabitants of great towns for domestic use may be universally adopted.

While this prophecy may have been fulfilled in England from the time when it was made, it is only approaching fulfillment in America. The first well-equipped plant for filtering water through sand in America was built at Poughkeepsie, N. Y., from plans prepared by the late James P. Kirkwood, Past-President, Am. Soc. C. E., and adopted by the city in 1869. This municipality had the honor of possessing the principal filtration plant of this kind in the United States for many years.

The work of the State Board of Health of Massachusetts, in regard to the purification of water and sewage, is well known, but I should leave out an important part of the development of water supplies and water-supply engineering, were I to omit it; moreover, the organization of this work gives me one of several opportunities to speak from my own experience of the advantages of cooperation between engineers and those engaged along



HOISTING TWO COMPLETE SETS OF UPPER CHORD BARS FOR ANCHOR ARM. EACH SET WEIGHS ABOUT 60 TONS. VIEW TAKEN NOV. 17, 1905.

of applied science. This Board, by a law passed twenty years ago, was given general supervision of the streams, water supply and sewage disposal of Massachusetts, and was required to advise cities and towns as to the appropriateness of sources of water supply, and as to the best method of disposing of the sewage.

Mr. Hiram F. Mills, the eminent hydraulic engineer, and then Chairman of the Committee on Water Supply and Sewerage of the Board, after learning how little definite information was then available on many phases of these subjects, began with wonderful foresight initiated the now classic examinations and experiments which have resulted in so great an increase in the knowledge possessed by the water-supply and sewerage engineers of the present day. In this work he was ably supported by the Chairman of the Board, Dr. H. P. Walcott, an eminent sanitarian. The speaker was at the time the Chief Engineer of the Board. For making these examinations and experiments a staff of chemists, biologists and bacteriologists was organized, comprising as leaders such men as the late Thomas M. Drown, Professor of Chemistry in the Massachusetts Institute of Technology, and afterward President of Lehigh University, and Professor William T. Sedgwick, of the Massachusetts Institute of Technology; and in the ranks such men as Allen Hazen and George W. Fuller, Members Am. Soc. C. E. While this is not an address on sewage purification, I cannot refrain from stating that the experimental work for the purification of sewage and water was planned and executed under the personal direction of Mr. Mills, who, although serving on an unpaid board, gave to this work the larger part of his time for several years.

The examinations of the water supplies of the state, with which the speaker was more intimately associated, and the continued analyses of the waters, proved the great value of the cooperation of engineers, chemists, biologists and bacteriologists, as was recognized by all who participated in the work. These advantages may not be as fully appreciated at the present day, when there is no difficulty in finding individuals who are well acquainted with the engineering, chemistry, biology and sanitary history of water supples, and who are in many cases experts along more than one of these lines, but men with such knowledge did not exist twenty years ago, and that they are available to-day in this country is to be credited to a large extent to the work of the Massachusetts Board of Health.

Continuing with the subject of the cooperation of engineers with those in other branches of applied science, I wish to express my indebtedness to a skilled geologist, who, from extended examinations at the surface of the ground, with the assistance of only a limited number of borings, predicted with a high degree of accuracy the character of the rock to be encountered in different parts of several aqueduct tunnels; who, in extensive boring operations assisted by his advice as to the best method of obtaining trustworthy samples of the material penetrated, and who, by painstaking work and the resources of the geologist, interpreted the results of some important borings more accurately than they could have been interpreted by the engineers.

It is perhaps unnecessary to refer to the importance of consulting an architect when important buildings are to be erected, because it is now the general rule to do so, but there are more exceptions to the rule than there should be.

It is less common for engineers to consult a landscape architect, as many have faith in their ability to produce pleasing results with geometrical forms. The results of such cooperation in my own experience have been particularly satisfactory, as the carrying out of a landscape architect's plan, made before beginning work, has resulted in pleasing effects at little additional cost.

The relative advantages of municipal and private ownership of public utilities is a live question in municipal circles at the present time, and in the discussion of this subject water supplies furnish the most valuable object lessons. In the days of Greece and Rome, the aqueducts and other works were built by the kings and emperors, and operated under their direction. The question of municipal ownership arose in more modern times, when the municipalities became wealthy and to a large extent self-governing. The Hampstead Heath Water Supply, for a portion of London, was built by the municipality and finished in 1590, but about one hundred years after, in 1692, it is recorded that the municipally owned water supplies taken from sources within five miles of London were sold to the Hampstead Water Company. After this sale London was supplied by private companies until 1901, when all the immense works for supplying water to that city were again taken by the municipality. In America the water-works of Philadelphia were municipally owned from the beginning, but the earlier supplies for New York and Boston were introduced by private companies.

There has been for a long period of years a movement toward the municipal ownership of water supplies, and most of the larger water-works in America and in Europe are so owned. The municipally owned water-works, as a rule, have been successful in all respects.

THE DESTRUCTIVE EXTENT OF THE SAN FRANCISCO EARTHQUAKE OF 1906.

By Charles Derleth, Jr., C. E.*

INTRODUCTION.

In "Engineering News" of May 17, 1906 (p. 548), I presented some hurried statements with typical photographs intended to bring promptly before the engineering public the nature and extent of earthquake and fire destruction in the immediate vicinity of the San Francisco peninsula.

Figure 1, p. 548 of that issue showed that a certain fault line, running approximately N. 35° W., passes very near to San Francisco, and the written matter indicates emphatically that the degree of destruction depended very decidedly upon the nearness of the place of observation to the line of fault. Since that writing I have had the opportunity to trace the fault for a distance of more than 200 miles, and I have examined all localities near it for the purpose of studying earthquake destruction to engineering works.

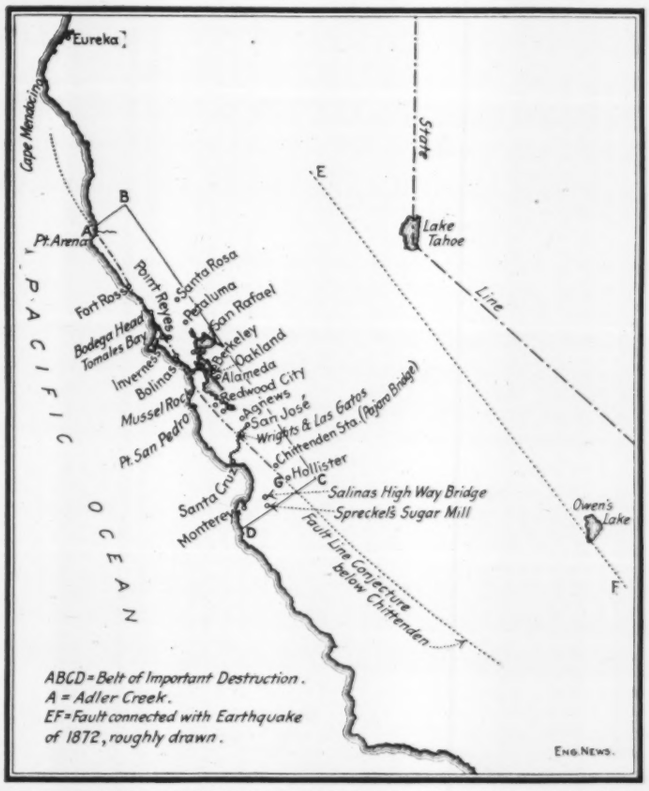


FIG. 1. MAP OF CALIFORNIA, SHOWING POSITION OF FAULT LINE; EARTHQUAKE OF 1906.

The crust of the earth is constantly adjusting itself to the conditions of stress and strain. The surface of the globe is gradually and slowly changing its form to suit these adjustments. The span of a human life is quite negligible in comparison to the geologic ages required to bring about marked deformations in the surface of the earth, and consequently many of us are not aware of the slow crustal movements which are constantly in evidence throughout the world's surface to the eye of the experienced geological observer.

Some parts of the earth's surface are slowly sinking. I believe the seacoast of the State of New Jersey is dropping. In other regions the land is rising. It is said that the coast ranges in California are young mountains and are pushing their way through the coastal plain. These are effects of the so-called mountain making forces which act through long periods of time. There is no such thing as the "everlasting hills." The phrase is a figure of speech.

Some parts of the earth's surface are more settled or stable than others, and we do not expect severe crustal movements in such regions, for example, New York City and vicinity; in other places crustal movements, that is, earthquake phenomena, are from time to time expected. Japan and the Pacific Coast of California are such regions. I am for the present not considering volcanic earthquakes. These are generally local and of smaller extent and are comparatively rare. The tectonic or mountain-making earthquakes are very much more frequent and may affect large surfaces, and while they are apt to occur more often in regions of proved crustal weakness and instability, they are liable to occur anywhere and at any time.

The crust of California, dynamically speaking, is alive and active, and I do not see any object to be gained by refusing to admit this fact. Earthquakes are natural and should not be feared. They offer to the geologist most interesting dynamical problems, and their effects, no matter how unfortunate, furnish to the engineer most valuable experience and counsel. The California earthquake of April 18 seems to me one of the grandest examples of earth movements that has yet come to the notice of civilized man, in these later days of precise instruments and more exact thinking, by the aid of which it is the privilege of present day geologists and engineers to study.

Nearly a generation ago, geologists had mapped out long continuous fault lines upon the face of California. These lines are the results of former slips or accumulations of slips that have occured in the past, often the remote geological past and thorough study reveals their relation to California topography. These faults are lines of weakness in the crust, and ruptures or movements in the rocks far below the surface naturally may produce surface evidences along these geological scarps.

Most of these important fault lines, if not all, run in a general north northwesterly direction, essentially parallel to the main mountain range lines, a parallelism which is quite natural and to be expected. The great earthquake of 1872 was accompanied by and was coincident with a heavy slipping along a fault line E F (See Fig. 1) which runs approximately from the region of Owens Lake in the south, toward Lake Tahoe in the north.

THE MAIN COAST RANGE FAULT.

The earthquake of April 18 has affected the surface along the main coast range fault. This fault A G (see Fig. 1) starts from the vicinity of Point Arena in the north, passes in a south-southeasterly direction along the coast through the region of Hollister in the south, and it is said by geologists that it may be traced almost to the head of the Gulf of California. Marked surface slipping on April 18 did not occur throughout this entire length, but appeared prominently only from Point Arena to Hollister, a distance of about 250 miles.

The writer followed and examined the fault from Point Arena to Hollister. Within these limits will be found the important destruction to engineering works. It is plain that the extent of the earthquake is large. The belt of chief de-

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Fig. 2. Redwood Tree Standing Right on the Fault Line, ½-Mile South of Fort Ross.

(The rift in the ground is seen in front of the crack in the tree's trunk.)

struction, A B C D, Fig. 1, covers an area of about 50 × 300 miles. It is probable that heavy earthquakes along this pronounced line of crustal weakness will not occur soon again in the future because the region has been put into equilibrium. But it is plain that thinking men must expect surface disturbances in California, and it is their duty to anticipate them. Anyone who has carefully examined the earthquake destruction cannot fail to admit that the great structural losses are due in too many instances primarily to fauty design, poor workmanship and bad materials, occurring, let us hope, through ignorance and a blind disregard for earthquake possibilities, yet I regret to add that I feel convinced that much of the bad work is due to a combination of criminal carelessness and cheap construction. Rather than try to tell outsiders that San Francisco was visited by a conflagration, I believe that it will do San Francisco and California in general more lasting good to admit that there was an earthquake and that with honest and intelligent construction and the avoidance of geologically weak locations our losses would not have been so great.

Referring again to Fig. 1, it is seen that the fault is almost a straight line. It disappears in the ocean at the extreme north a few miles above Point Arena at the mouth of Adler Creek. It probably does not again cut the land north of Point Arena, although some geologists think it may possibly curve toward the east and be again found for a short distance along or near the shore at Cape Mendocino, because the western part of Humboldt County was severely shaken, but I have heard of no accurate reports from that region, and I have not personally visited it.

From Adler Creek the fault may be followed southward approximately parallel to the coast line to Fort Ross, where it runs into the ocean again about two and one-half miles south of the Fort. It is interesting to note in passing that Fort Ross is an old Russian fur trading post, settled in 1811. The Russians occupied this post for about 30 years. Their old timber buildings still remain, in considerable decay. This explains geographical names in the vicinity, such as Russian River, and Sebastopol. Earthquake vibrations were very severe in the neighborhood of Fort Ross; I believe more severe than in the vicinity of San Francisco, but there were no important human structures to demolish.

Figure 2 shows a redwood tree 6 ft. in diameter which happened to stand right on the fault line near Fort Ross, and whose trunk was split into two halves for a distance of 35 ft. upwards from the ground. The westerly half was sheared toward the north and actually moved past the east half a distance of about 8 ins. Many trees have been more or less ruptured along the fault line where it traverses the forests, and in one place I observed leaning trees for a considerable distance, distinctly marking the location of the fault through the forest. Leaning trees make a very uncommon and awe-inspiring sight in a redwood forest, where the trees are over 200 ft. in height. At Fort Ross, near the fault, I noticed many partly decayed trees snapped and demolished by the shake, although sound ones near at hand were undisturbed. I could not help thinking how nature in this way showed her weak and her strong construction. In the cities that I have visited, from Santa Rosa to San Jose, one can pick out the black from the white sheep, in buildings. The works of nature and of man are tested alike.

DIFFERENTIAL HORIZONTAL DISPLACEMENT.

From Point Arena to Hollister all evidence clearly indicates that the ground on the west of the fault line moved north on an average of from 7 to 9 ft. relative to the ground on the east of the fault line. Straight fences that crossed the fault have been invariably sheared or disturbed so that they are out of alinement or offset from 6 to 15 ft. Everything that crossed the fault (water pipes, houses, dams and water courses), has been sheared. It is clear to the observer that the ground on the west of the fault moved; that on the east did so too, probably the material along the fault moved in opposite directions on the two sides with the resultant displacements mentioned. The tearing of the surface along the fault also clearly shows that there was some torsion.

In my previous article relating to the Spring Valley Water Company and the vicinity of San Francisco, I mentioned fence offsets of 7 ft. near San Andreas reservoir. One offset near Fort Ross measured 15 ft. 6 ins. Figure 3 shows a fence one-half mile south of Fort Ross which did not collapse but was curved and bent to suit the lateral movements of the earth so that its unmoved parts are now 9 ft. out of alinement. A roadway in the Point Reyes locality was dislocated about 20 ft.

The amount of offset along the fault at the surface is affected by the nature of the surface material. In marshy and sandy ground and alluvial valley lands it is sometimes more and sometimes less than the average—due to secondary motion of the looser surface materials. Loose surface ground was shifted and marshes shook like bowls of jelly, even when distant from the fault line. It would seem that there was somewhat more—perhaps 2 ft. more—sliding motion along the fault line at its northerly end near Fort Ross than at its southerly end near Pajaro, where the movement diminishes. Throughout the disturbed region, latitudes and longitudes have doubtless been changed a few feet.

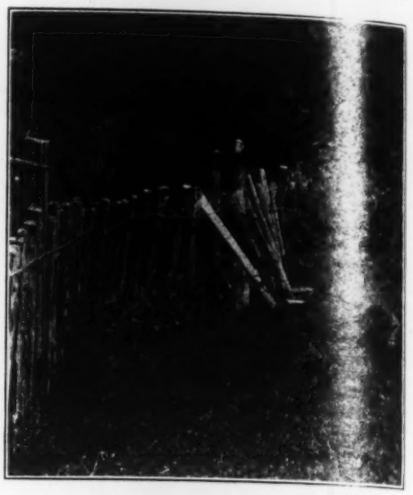


Fig. 3. Fence ½-Mile South of Fort Ross. Bent and Offset 9 Feet.

VERTICAL DISPLACEMENT.

There is little evidence of change in elevation of the land and only careful levelling and comparisons with geodetic records can throw exact light upon this question. There has been some vertical as well as horizontal displacement throughout the disturbed region, and mountain tops probably have been changed in elevation by a few feet, although it may never be possible to prove such a statement.

The relative horizontal movements along the fault are much more marked than the differential vertical displacements. In the north, in Sonoma County, I observed near Fort Ross, a general lifting of the surface to the west of the fault line. The amount does not exceed 4 ft., and, I believe, is general in the northern region. In the south there is little if any differential vertical displacement.

The fault line is lost in the ocean for some miles south of Fort Ross. It is again noticed, as one proceeds southward, where it crosses Bodega head and cuts through a sandy spit which extends into the ocean. At this point its surface effects are very indistinct and of little moment to the engineer.

Again the fault line disappears in our southward journey, but it distinctly follows through the structural trough which forms Tomales Bay, and appears on land again at the southerly end of that bay near Point Reyes and Olema. From Point Reyes to Bolinas Lagoon the fault line is

FIG. 4. WRECK OF COURT-HOUSE DOME, SANTA ROSA.

(The picture is taken from the roof of the second story.)

FIG. 5. CARNEGIE LIBRARY BUILDING, SANTA ROSA.
(A stone faced building with interior brick walls and wood framing.)

FIG. 6. MEMORIAL ARCH, STANFORD UNIVERSITY, PALO ALTO.
(Showing large masses of brick work which have fallen 100 ft. without disintegrating.)

extremely distinct, and this region offers to the geologist and the engineer equally interesting evidence. At Bolinas the fault again disappears into the ocean outside of Golden Gate, and does not appear on land again until we reach Mussel Rock on the San Francisco peninsula. Figure 1 of my previous article (p. 548), plotted the fault line slightly in error in that it shows the line curving from Mussel Rock along the coast of San Francisco peninsula to Lake Merced and thence to Bolinas. The line should be straight from Mussel Rock to Bolinas. The great coastal disturbances from Mussel Rock to Lake Merced are landslides on steep banks, produced by the nearness of the shore line to the fault. Heavy landslides are also found on the coast to the south of Mussel Rock near San Pedro Point and Devil's Slide, where the preliminary grading for the Ocean Shore Road which is to connect San Francisco and Santa Cruz was entirely wrecked.

From Mussel Rock the fault proceeds southward through the valleys of the Spring Valley reservoirs of San Andreas and Crystal Springs, as already explained. It then continues southward somewhat to the west of Redwood City and Palo Alto (7 miles distant) and crosses the narrow gage railway, which runs from San Jose to Santa Cruz, in the neighborhood of Los Gatos, where a number of mountain tunnels on the railroad have been made impassable by slides, so that trains up to the present are not able to cross through the Santa Cruz mountains. San Jose and Agnews are thirteen and twelve miles, respectively, from the rift.

The fault line may then be clearly followed southward to the region of Chittenden Station on the Southern Pacific R. R., where it crosses the Pajaro River about 1,500 ft. south of the station and passes right through the Pajaro railroad bridge, to which I will refer later. The fault line then proceeds inland toward Hollister, where it leaves the region of important structures, and beyond which point I have not traced it.

EFFECT OF GEOLOGICAL STRUCTURE.

Within the belt of destructive effects above outlined, the intensity of shock varied greatly. The maximum destruction is found on the fault line, where no structure, no matter how well built, unless very fortunately placed with respect to the rift line, could withstand the attack. Next in order of severity we find marshy lands and artificially filled ground, for example, the San Bruno marsh and parts of the Mission district of San Francisco. A line of considerable but lesser intensity of shock may be traced approximately parallel to the fault line through the bottom lands of the valleys which contain Santa Rosa and Healdsburg in the north, and Agnews and San Jose in the south, with the Bay of San Francisco as a central feature. These bottom lands are alluvial deposits of soft and considerably incoherent materials. Such surface materials have everywhere been markedly shaken. It is a general observation that the earthquake waves transmitted by such formations appeared to be much more destructive than waves which traversed the hard and more elastic rocks. An interesting problem is here presented for mathematical treatment, namely, what happens when an earthquake wave of high period and small amplitude passes from a hard and elastic material into a soft granular mass? It would appear that the wave is transformed into one of longer period and larger amplitude. The billow-like effects that appear in the streets near the Ferry House in San Francisco are most excellent examples of the resulting deformation in soft incoherent materials.

These observations seem to offer an explanation for the severity of the shock in cities like Santa Rosa and San Jose, situated in the bottom lands of valleys, when compared to places like San Rafael, Petaluma and Berkeley, which rest on much firmer foundations. In San Francisco the destruction on the rocky side hill slopes of the city is the least; on the softer ground, the sand dunes, and the artificially filled areas, the destruction is increasingly greater in the order named. In the south, on the west side of the fault, the valley of the Salinas river was badly disturbed. It is a soft river deposit. In striking contrast the city of Santa Cruz nearby was comparatively unharmed; it rests on a rocky and more stable foundation.

FIG. 7. SAN JOSE HIGH SCHOOL BUILDING.
(This structure was so badly wrecked that it was completely taken down.)

SANTA ROSA.

Santa Rosa, about fifty-two miles north of San Francisco on the California & Northwestern Ry., is nearly twenty miles east of the fault line. Nevertheless it has been visited by great earthquake and fire destruction. Eastern people hear little of the losses of Santa Rosa because they have been overshadowed by the largeness of the destruction at San Francisco; yet in my judgment, proportionately speaking, Santa Rosa's loss is greater than that of San Francisco. The city stands on alluvial ground. Its business center has been wiped out by fire and practically every building in the business district completely collapsed in the earthquake. I believe all but two or three of Santa Rosa's brick buildings were razed to the ground by the temblor; but it is my judgment that the shock was less serious in the northern city than at San Francisco.

How then should the general destruction be explained? The brick buildings of Santa Rosa were carelessly constructed. Lime mortar was almost invariably used with bad brick bond. The sand of the lime mortar used was what is locally known as "drift" sand, containing, according to a local engineer, practically 50% of loam. When we remember that it has been too common a

FIG. 8. REAR WALL, HALL OF RECORDS, SAN JOSE.
(Showing how the heavy granite faced walls ripped away from the interior.)

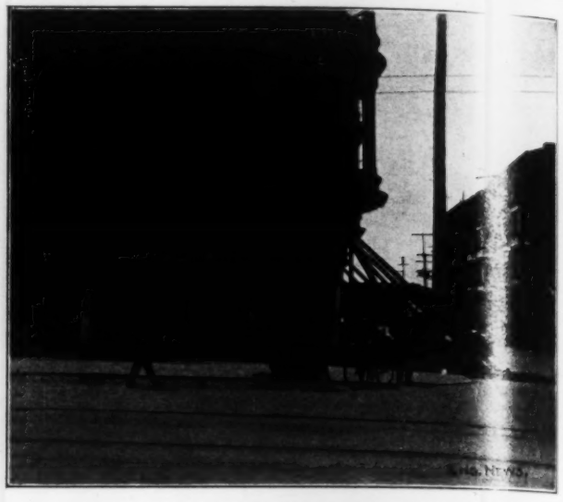


FIG. 9. VIEW OF FIRST STREET, SAN JOSE.
(Buildings with little lateral bracing in first story.)

custom in California to lay brick for small structures without sufficiently wetting them, in fact, almost dry, what else should be expected, especially when we further observe a most inadequate anchoring of the floor and roof frames to the outer walls, and a usual absence of necessary cross walls or frames.

Wooden buildings in Santa Rosa with few exceptions were unaffected by the earthquake except for the general throwing down of chimneys. The few frame houses which were wrecked, collapsed because of faulty or rotten and improperly braced underpinning to the first floor or cellar.

No railway or highway bridges in the vicinity of Santa Rosa suffered. No breaks occurred in the water-works system, its artesian sources of supply, or its pumps. A few house service pipes broke at the house lines. No sewers were crippled. It is plain to me that there was little or no unequal resultant movement of the ground at Santa Rosa such as one finds at the fault line. The destruction was due merely to earth vibration and the brick buildings almost without exception collapsed like so many sand piles. Figures 4 and 5 give typical Santa Rosa views. The business portion of Santa Rosa burned because fires started simultaneously in the rubbish and the facilities for fighting fire were too limited to cope with a conflagration. There was never a lack of water; in fact there was more than under normal conditions because the people were not using their natural supply, and for the same reason the pressure in the mains was greater than under normal conditions, although some local Santa Rosans ascribe the higher pressure to the effect of the earthquake on the subterranean grounds surrounding the artesian wells.

PALO ALTO.

Palso Alto is seven miles east of the fault line. It is unnecessary to describe in detail the destruction at Stanford University or to append photographs, since much information has already been given to the public through the medium of the press.

I have visited Stanford University. The extent of destruction is great. The grade of workmanship, in my judgment, is of relatively high quality for California construction. I attribute the excessive destruction to three causes: (1) the nature of the ground, (2) nearness to the fault line, and (3) the type of construction. I consider the third reason by far the most important. Heavy stone-faced buildings of the type we find at Stanford, no matter how well built, in my opinion cannot resist an earthquake of the intensity experienced at Palo Alto, when we consider the nature of the ground upon which the buildings rest. Everywhere on the campus we find top-heavy arcades and walls supported by light stone pillars. The figure on p. 634 of Engineering News for June 7 1906, gives an emphatic example. Roofs in general are heavy and covered with tile. The interior framing of necessity lacks the unity and stiffness which can only come through steel cage construction or the best types of reinforced concrete cage construction. Considering earthquakes I do not think the type of structure at Stanford has been happily chosen. The Stanford buildings represent a type which lacks unity and coherence of frame. At the same time they are very heavy. The fault is in the type of design, and not in the execution, and I see no reason for any personal criticism.

I have seen many pictures of the Stanford Memorial Arch showing dramatic destruction. These pictures I saw before visiting Palo Alto and they gave me a false impression. Fig. 2, p. 509, in Engineering News of May 10, 1906, shows that the arch ring itself has not been thrown down. In fact there are but a few cracks in the crown of the soffit. The ring stones have hardly been moved, if at all. It was the heavy top and cornice, insufficiently tied, that fell; as they should. It would be surprising had they not fallen. Above the arch was a large box-shaped mass of masonry upon which rested the heavy overhanging cornice, and this box had practically no transverse framing or partition cross-walls of any kind. The masses that dropped from the arch fell in large pieces nearly 100 ft. to the pavement without crumbling (see Fig. 6), which proves that the mortar was good. Considerable cement was used in the mortar for the Memorial Arch. Perhaps not so much, but at least a goodly amount of cement, was used in every building which I examined.

In my opinion, had Stanford's buildings been of reinforced concrete, or of a good type of light brick or stone construction bonded to a rigid metal frame, the destruction would have been far less. The type of building to withstand earthquakes should emphasize unity and lightness, combined with rigidity and elasticity.

SAN JOSE.

San Jose, about 40 miles to the south and east of San Francisco, is 13 miles to the east of the fault line. This city fortunately was spared a conflagration. The earthquake destruction is appalling, as is shown by a few typical photographs (see Figs. 7, 8 and 9). San Jose's water supply system, like that of Santa Rosa, was not injured; its sewers also are intact, showing that there was no unequal displacement of the ground. The earthquake destruction was generally the result of earth vibration and poorly constructed buildings. Again we find cheap construction with lime mortar, and weak framing and anchoring for floors and roofs. Whole sides and fronts of two and three-story store and office buildings on the main business streets collapsed. On First St.,

FIG. 10. AGNEWS INSANE ASYLUM.

FIG. 11. GENERAL VIEW OF PAJARO BRIDGE.

in the main business center, in one instance a row of about six or seven buildings careened, due to a lack of sufficient transverse framing in the first story. Stores occupied these main floors, demanding large window openings and as much floor space as possible. Virtually the upper portions of the buildings rested on stilts (see Fig. 9). Public buildings in San Jose with the exception of the Postoffice were generally racked. (The tower of the Postoffice fell.) In too many instances we find heavy stone facings improperly anchored to the interior frames, and roofs without system. In the great area of destruction one finds roofs depending for the support upon the stiffness of the walls upon which they rested, while from the nature of design and construction the walls lacked stiffness. These flimsy or clumsy roofs, when subjected to earthquake vibration, tended to lower in the peak and to spread at the ends. Naturally they crushed out the weak outer walls, lacking in bond, and inherently weak in mortar. Every public building, most school houses, and many churches in San Jose, where built of stone or brick, have been demolished or severely damaged. Some of these structures still stand, but if wisdom is exercised many of them will be torn down. Even some frame buildings, notably churches and the annex to the Hotel Vendome, improperly framed and with weak underpinning, completely collapsed.

AGNEWS ASYLUM.

Six miles to the north of San Jose one finds the Agnews Insane Asylum, consisting of one large brick timber-framed building about two city blocks in length, surrounded by some twenty similar but smaller brick structures. One hundred patients and eleven attendants, I believe, were killed on the morning of the earthquake. I do not know how many were injured. Not one of all these buildings is now habitable. Of all the destruction that I have seen—and I have visited the whole disturbed area—this cluster of buildings exhibits the most complete earthquake destruction, with the possible exception of the City Hall in San Francisco. They are both public buildings. Is it not time for California to seriously realize the situation? The main central brick structure and tower of the Agnews Asylum (see Fig. 10) entirely collapsed. The central tower fell en masse and crumbled to pieces so that three distinct wings of the building, themselves much demolished, and left disconnected. Some of the outer and smaller buildings have 9-in. brick walls for a height of two stories or more. It is significant to observe that a high water tower of structural steel, situated close to the power house of the asylum, is entirely intact. The power house, built of brick, and its high brick chimney, have collapsed with the rest of the brick structures.

PAJARO BRIDGE.

The Pajaro bridge (Figs. 11, 12, 13 and 14), is perhaps the most interesting structural example of violent earthquake effect. The bridge is on a 6° curve whose chord makes an angle of about 10° with the fault line. The bridge is right on the fault. In consists of a deck plate-girder span of about 50 ft. at each end and four deck Pratt trusses, each of about 120-ft. span. The abutments and the intermediate piers are of concrete with granite copings. This structure in my judgment would not have been affected by the earthquake had it been at a reasonable distance from the fault line, but it was subjected to a most severe racking, since the ground upon which its piers rest moved unequally. I believe some portions of this bridge were moved several feet more than others, but exact information should come from the engineers of the Southern Pacific Ry.

Figure 13 shows the south abutment pier 1, where the steel plate girders have been moved 24 ins. off the abutments and are now supported by a temporary wooden bent. Fig. 14 shows the first intermediate pier, No. 2, at the south end of the bridge and the two adjoining spans, the one a plate girder and the other a Pratt truss, temporarily supported by timber falsework. This pier near ts base has been ruptured in a horizontal section, probably in a level between two layers of concrete. The mass of the pier above this rupture has been moved northward about 3 ins. as is shown in the sketch (Fig. 12). The coping has been ripped from the top and moved northward more than 20 ins. The segmental rollers of the steel pedestals have been smashed and the anchor bolts severely bent. The next intermediate pier, No. 3, to the north (see Fig. 12), has also been cracked near its base and the upper portion moved some 2 ins. The pier leans somewhat to the north, and the anchor bolts above the coping have been bent so that the pedestals of the steel work are moved some 3 ins. The distortion grows less as one approaches the northern end of the bridge. The northern abutment is considerably cracked. The whole structure seems to have been moved somewhat to the east and is slightly out of plumb. The earth motions are clearly seen in Fig. 13 at the south abutment, where the sloping earth has been

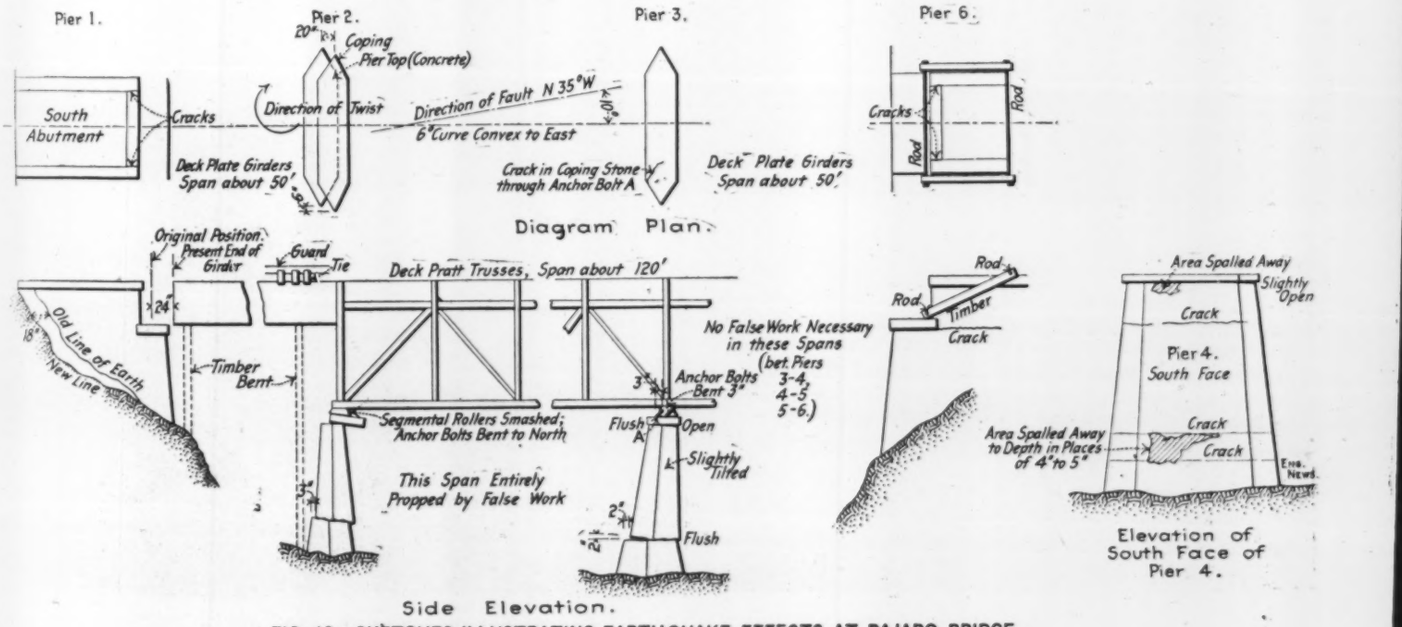


FIG. 12. SKETCHES ILLUSTRATING EARTHQUAKE EFFECTS AT PAJARO BRIDGE.

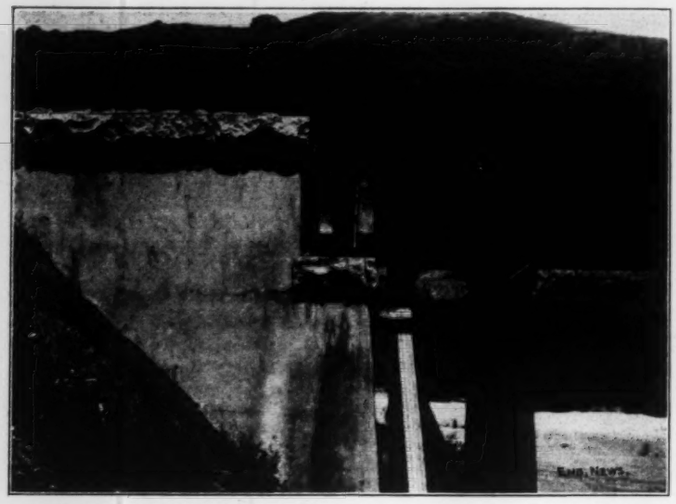


FIG. 13. SOUTH ABUTMENT OF PAJARO BRIDGE.

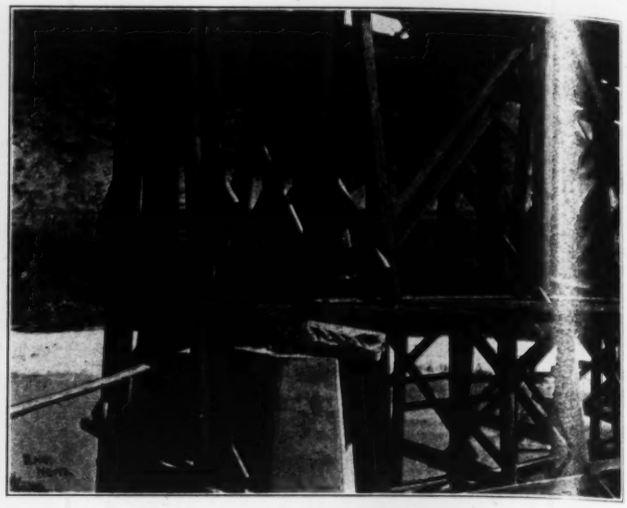


FIG. 14. PIER 2 OF PAJARO BRIDGE.

moved 18 ins. along the side wall. Fig. 11 gives a general view of the bridge.

SALINAS HIGHWAY BRIDGE.

The region of Salinas is considerably to the west of the fault line, but the Salinas river runs through a marshy district of river deposits with small elevation above the sea level. Although this region is at a distance from the fault line, there is much evidence of surface movement (as distinct from elastic vibration), as in many other instances of soft surface deposits, for instance, the San Bruno marsh near South San Francisco, where the Crystal Springs pipe line of the Spring Valley Water Company was so badly ruptured. The land on the south bank of the Salinas river for a considerable area has moved into the river in a northerly direction, on an average, through a distance of about 6 ft. Figs. 15 and 16 show the south abutment of the Salinas highway bridge. The ground has moved under the superstructure about 6 ft., bending the pile bent foundations as shown in the pictures without seriously injuring the trusses. A 3-in. oil pipe-line crossing the bridge was ruptured on the south approach, one length of pipe being bent like the letter "S." The northern end and approach to the bridge were hardly affected. In fact, the northern bank of the river shows little disturbance at this point. From the Salinas bridge eastward a distance of about 1½ miles to Spreckels, the south bank of the river is continuously scarred and rent so that the old road is impassable for vehicles.

SPRECKELS' SUGAR MILL.

At Spreckels is found the Spreckels' sugar mill (Figs. 17 and 18). These buildings are of excellent construction. The main building, about 100 ft. × 500 ft., a huge box-shaped structure with heavy steel frame, brick walls supported on the frame, and arched concrete floors with steel floor framing, rests on a pile foundation. The mortar for the brickwork is excellent and contains considerable cement. Had these buildings been in Santa Rosa or San Jose, I think they would have stood the earthquake with possibly a few slight cracks and the loss of some bricks here and there near the cornices. But unfortunately they rest on soft ground, very much like the made ground areas of San Francisco. Considerable portions of the brick walls have been thrown out of the steel frames. The long walls of the main building are supported by posts about every 15 ft. and are braced transversely at these posts by the interior framing, except in the middle third of the building, where there is an open space from ground to roof to give clearance to the huge machinery. In this middle third, where the long walls are not laterally braced by the transverse frames and floors, the walls have been badly buckled. The east wall is buckled concave outward. The brick work is badly cracked, but little of it has fallen. The west wall, in the middle third, is buckled convex outward and naturally the brick work has been thrown down and outward in large masses. Fig. 17 shows the west wall of the building. I believe this building was subjected to an earth movement from south to north, which accounts for the buckling of the long walls in the middle unsupported third. There may have been also some settling. Large masses of brick in the north wall (Fig. 18) have been thrown down. There is also a rupture in the carriage way to the dumps, clearly showing the existence of a decided ground movement at the time of the earthquake. All the smaller buildings have been damaged to some extent. The Spreckels' Sugar Mill was well designed. It was probably subjected to a more severe racking than any other well designed structure in the State. It can be readily repaired.

CONCLUSIONS.

There has been much comment of late regarding the relative merits of sea level and lock types of canal for the Isthmus of Panama in the light of earthquake possibilities. It is my judgment, from what I have seen of the California earthquake, that honestly, scientifically and generously constructed works can abundantly withstand any reasonably severe earthquake shock so long as the structure does not happen to be located on a fault line or a place of very unequal motions of the ground. Even in such severe locations it is seen that the 93-ft. San Andreas earth dam still fulfills its functions. The Pajaro bridge, though badly crippled, was speedily made usable, and the Spreckels' sugar mill can be repaired without relatively great expense.

In the future, for important works, such as water supply conduits, engineers should avoid fault lines and marshes. Dams should not cross fault lines, and when possible should not run at right angles thereto; and yet, considering their purpose, it may be necessary to place dams in the most unsatisfactory positions. I do not consider it wise to encourage in California the building of bold arch dams of thin construction. It would be better to place important railway

FIGS. 15 AND 16. VIEWS OF SOUTH ABUTMENT OF HIGHWAY BRIDGE AT SALINAS.

FIG. 17. SPRECKELS' SUGAR MILL, MAIN BUILDING, WEST WALL.

FIG. 18. SPRECKELS' SUGAR MILL, MAIN BUILDING, NORTH WALL.

bridges at river crossings not coincident with lines of crustal weakness or upon soft alluvial deposits; and where structures must be built upon treacherous ground or near fault lines, no expense should be spared for good materials, high grade workmanship and intelligent design. Generous factors of safety should be used and the best designing talent employed. California engineers should pay some attention to geological structure.

ANNUAL CONVENTION OF THE AMERICAN RAILWAY MASTER MECHANICS' ASSOCIATION.

The most successful convention in recent years of the American Railway Master Mechanics' Association was held at Atlantic City, N. J., on Monday, Tuesday and Wednesday of last week—June 18, 19 and 20. In the number of railway motive power men present, in the number and character of the exhibits, in the quality of the papers and discussions, and in general attention to convention work, the convention reached a high standard of attainment. As an indication of the facts, some 400 members were registered, and to them must be added the names of at least thirty-five hundred guests; to the previous list of exhibitors there were added 50 firms dealing in railway supplies. Of the character of the papers and discussions the succeeding paragraphs will give some indication.

BUSINESS PROCEEDINGS.

In his address opening the work of the convention Mr. H. F. Ball, President of the Association, reviewed briefly the advances of the year in locomotive practice. The tendency had been to work along improvements in details, no radical changes in general design being noticeable. The four-cylinder balanced compound had not progressed in general favor as rapidly as many had anticipated, and in view of the economy obtained in the tests at the St. Louis testing plant it certainly seemed deserving of more attention. The Mallet type of locomotive was another which was not being introduced as fast as its merits warranted. In detail design the most noteworthy departure was the general acceptance of the Walschaert valve gear. This form of gear lent itself admirably to heavy locomotives, having advantages in the way of accessibility for inspection and repairs, lightness and freedom from rapid wear. In boiler design much had been done incidental to the introduction of the superheater. Apart from this, the most radical departures in boiler design were found in the use of a combustion chamber in long boilers using bituminous coal, with attendant remarkable results in overcoming rapid deterioration of flue ends and leaking. It was also worthy of note that the flexible staybolt was steadily gaining ground. It appeared questionable to look for further economies as a general proposition by continuing to enlarge the locomotive under present conditions. This development had been arrested by the limitation of clearances, capacity of the fireman and the reaching of the practicable train length limit. The solution of the problem lay in other directions. We shall be required to develop the mechanical stoker; compounding and superheating would be prosecuted with greater vigor. The use of a feed-water heater, of compound air pumps and of variable exhaust nozzles, perhaps, were other items.

Following the President's address was the usual routine work of receiving communications and appointing committees on auditing of accounts, drafting of resolutions, etc. The secretray's report showed a total membership of 834; the treasurer reported a balance on hand of $1,870. It was voted that the dues for the ensuing year should be $5 per member. Notice was given of several proposed changes in the constitution of the association; vote on these changes will be taken a year hence. The changes proposed were, in brief, to have officers nominated by a nominating committee and not balloted for in convention until selected by majority vote; and to provide for representative members one for each railway company which so desires to be officially represented.

TECHNICAL PROCEEDINGS.

The technical proceedings of the three days' sessions were so extensive that only those parts of most general engineering interest will be given at length. The papers and reports of special interest to motive power officials only will be given brief mention by title, with a statement of the official action taken at the convention in respect to them.

DESIGN OF WHEEL CENTERS.—A committee at the last convention reported on the shrinkage allowance for tires of large diameters for use on cast steel wheel centers. The allowance recommended was $^1/_{80}$-in. per foot diameter for centers under 66 ins. and $^1/_{90}$-in. per foot diameter for centers over 66 ins. This committee was appointed to recommend a design for centers, and its recommendations were, in brief, as follows: Number of spokes to equal diameter of center in inches, divided by four; if the remainder is one-half or over, add one spoke; sectional area of spokes to be 9 to 10 sq. ins. at large end and 5¾ to 6 sq. ins. at small end; rims should preferably be cast solid without cores; the sectional area should be approximately 0.45 of sectional area of tire when new; the rim should be cast, when practicable, without shrinkage slots; diversity of practice makes it undesirable to recommend any particular form of retaining ring; tire should have a lip on the outside to prevent center being forced through it. Accurate measurements of tires, especially tires less than 2½ ins. thick, show that they "roll out" or lengthen circumferentially in service; this accounts for loose tires more than does permanent set in the centers.

In the discussion Mr. G. W. Wildin (Erie) and Mr. David Brown (D., L. & W.) called attention to the added difficulty of shimming tires designed as recommended with an outside lip projecting down over the center; shimming would have to be done from the inside of the driver, and in modern engines the space here was so occupied as to give little room in which to work. In reply to a query by Mr. Geo. L. Fowler whether the "rolling out" action mentioned was any different in high carbon and in low carbon tires, reply was made by Mr. S. T. Park (Mich. Cent.) that two makes of tires were used by his company, and both showed the action. Mr. G. W. Wildin (Erie) considered a lip or ring unnecessary to retain tires on centers and because of difficulty of shimming the tire recommended, urged that the lip be taken off before it was adopted as standard. Mr. H. H. Vaughan (C. P.) stated that he had been using the tire recommended, and no serious difficulty was found in shimming it when necessary from the inside. Continuing, Mr. Vaughan said that experiments made by him indicated that their rims often collapsed in shrinking on the tire. He advocated a heavier rim and a spoke of pear-shaped section, located so that the heavy section came directly under the line of tire thrust.

SPECIAL VALVE GEARS.—An individual paper by Mr. C. J. Mellin (Am. Loco. Co.), on "Special Valve Gears for Locomotives," described the construction and action of the Gooch, Allan, Hackworth, Joy, Walschaert, Helmholtz, Allfree-Hubbell and Young valve gears. The paper was wholly descriptive in character and was chiefly valuable for assembling in one place in concise language such a record of the various valve gears likely to come up for consideration in locomotive design.

The discussion was opened by Mr. J. H. Manning (D. & H.), who presented partial figures of a test made on a seven-mile 37-ft. ruling grade section of road with engines 131,000 lbs. on drivers equipped, respectively, with Allfree-Hubbell, Young and Richardson gears. The figures were as follows: Tonnage of trains, 1,639, 1,640 and 1,664; steam pressure, 190, 193 and 195; fuel consumption per ton-mile, 22, 22.7 and 23.3 lbs.; drawbar pull, 25,638 lbs., 24,175 lbs. and 23,150 lbs.; weather and wind conditions, about identical. Mr. W. L. Kellogg (Pere Marquette) stated that his road had had four Allfree-Hubbell engines in service since 1903, and from tests made found them to be the most economical engines in service. The road was equipping a fifth locomotive

with the same gear. Mr. I. C. Hubbell (Pennsylvania) stated that he had Walschaert gears on several consolidation and Atlantic type engines, and they had proved eminently satisfactory. Mr. Angus Sinclair thought that the Joy gear had received scant attention in America, considering its wide favor abroad. As to the Walscharet gear, there was something peculiar in the way people were taking it up now, when he considered that it was first tried 30 years ago and condemned. In reply to Mr. Sinclair, Mr. G. W. Henderson thought that the change in favor of the Walschaert gear from 30 years ago was readily accounted for by the fact that the space inside the wheels was in modern locomotives so taken up that an outside gear was practically necessitated.

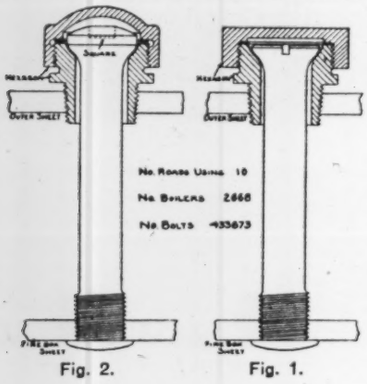


Fig. 2. Fig. 1.

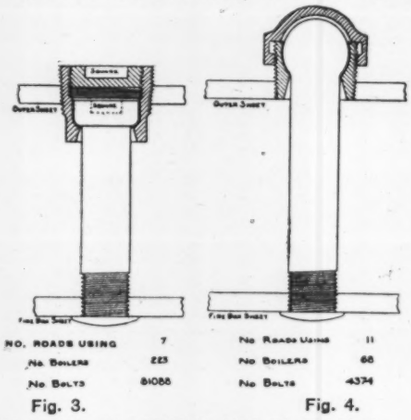


Fig. 3. Fig. 4.

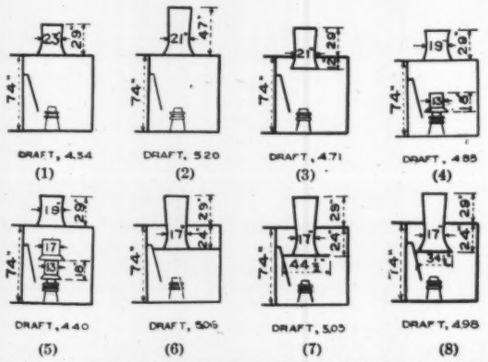
FIGS. 1 TO 4. DESIGNS OF FLEXIBLE STAY BOLTS IN GENERAL USE IN AMERICA.

FLEXIBLE STAYBOLTS.—One of the best reports of the convention was that on flexible staybolts. Beginning with a brief history of the device, which the committee finds to have originated probably with Mr. Wehrenfennig, of the Northeastern Ry. of Austria, in 1878, the report described the several forms of flexible bolts now in most common use and pointed out their weak and strong points, method of application and results in service. The report concluded with the results of a series of laboratory tests on flexible bolts.

In general, the committee found a growing appreciation and rapidly increasing use of flexible bolts. The designs that had been most used were those shown by Figs. 1, 2, 3 and 4. The design Fig. 1, which was the earlier one, developed serious defects, traceable to the screw slot, and the design shown in Fig. 2 had largely superseded Fig. 1. The design shown in Fig. 3 was a further development, and apparently a decided improvement. The design shown by Fig. 4 was the latest to be offered for use, and had met with considerable favor. So far, flexible bolts had been used in general only in locations where rigid bolts would not stand. Incrustation had been the principal difficulty that flexible bolts had had to contend with, and this was the cause assigned for most of the failures being reported.

The discussion on the report was mostly a citation of experience with flexible bolts. Mr. J. F. Walsh (C. & O.) had begun three years ago using flexible bolts in locations where rigid bolts had been failing most often. Altogether about 15,000 bolts were now in use in 210 to 215 engines. Of this number about 2,500 were of the design shown by Fig. 1, and the remainder of the design shown by Fig. 4. So far, only 23 bolts of the 15,000 had been removed for failure, and these were all bolts of the design shown by Fig. 1. He paid for the first bolts used about 90 cts.; now the cost ran about 35 cts. Mr. A. E. Mitchell (L. V.) had on Wooten boilers 40,000 bolts of the design shown by Fig 4. The first of these bolts were put in about two years ago, and so far not a failure had been reported. Mr. D. J. Durrell (P. R. R.) had many bolts of the design shown in Fig. 1 in service, and where treated water was used there had been no failures. Remarks approving the use of flexible bolts were also made by Mr. Wm. McIntosh (C. R. R. of N. J.), Mr. G. W. Wildin (Erie) and Mr. C. H. Doebler (Wabash).

LOCOMOTIVE FRONT ENDS.—The report of the Committee on Locomotive Front Ends deserves particular notice, not only for being an excellent piece of work in itself, but because it may be said to conclude a task which has been worked on almost continuously by the association for the last ten years. In 1896 a committee reported the results of a series of front end tests made at the locomotive testing plant of the Chicago & Northwestern Ry. These tests gave as a general result directions for the proper proportioning of exhaust nozzles. A second series of tests followed in 1902-3 at the laboratory of Purdue University. These results were expressed in the form of a general equation for the correct proportioning of stacks. In 1905-6 the third and last series of tests were made at Purdue. In brief, these last tests confirmed those of 1902-3 in respect to proper proportioning of stacks, and gave new data on the design of inside stacks and draft pipes. It was found that when the outside stack can have a height of 29 ins. above the smoke-box the extension of the plain stack into the smoke-box did not materially improve the draft. The use of a false top 24 ins. from the top of the smoke-box showed a material improvement in draft values over those obtained from either the plain inside or plain outside stack. Because of practical objections to the false top, experiments were made with plain annular disks at the end of the inside stack, and it was found that such an arrangement gave results substantially the same as those obtained from the tight false top. The development of this idea resulted in a design of an inside stack having a trumpet-shaped bottom, and it was found that this also gave results, with 34½ ins. bottom diameter, practically as good as did the false top. Experiments with single and double flounced draft pipes showed conclusively that equally good results could be obtained without their use. As a final result, the committee gave simple equations for the design of complete smoke-box draft appliances, and presented a drawing showing the shape of what it regarded as the best arrangements of locom[otive] ends. This drawing is reproduced in [this] may be explained as follows: Diagra[m 1] the best arrangement of plain stack, [in]side; diagram 2 shows best arrangem[ent] outside; diagram 3 shows best arrang[ement] the stack penetrating the smoke-box; [4] shows the best arrangement with [plain] pipe; diagram 5 with double-draft pip[e]; 6 shows the best arrangement with [double-draft] diagram 7 best arrangement with flat [plate] bottom of the stack, and diagram 8 bes[t arrange]ment with belled stack; diagram 9 [shows] arrangement of front end.



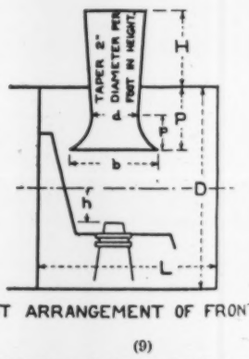


FIG. 5. DESIGNS FOR LOCOMOTIVE FRONT ENDS RECOMMENDED BY COMMITTEE OF THE AMERICAN RAILWAY MASTER MECHANICS' ASSOCIATION.

The discussion of the report consist[ed] mostly of questions from the members and ans[wers] from the committee explaining and reiterat[ing] statements in the report.

LOCOMOTIVE LUBRICATION.—The [Com]mittee on Locomotive Lubrication presente[d] the following general conclusions: For locomo[tives] with steam pressure as high as 225 lbs., or th[ose] using superheated steam the temperature of [which] is as high as 600° F., the ordinary valve oil has been found by experience to be quite satisfactory. The problem is to deliver the oil in proper quantities to the place where needed. For internal lubrication 75 miles per pint for large freight locomotives and 80 miles per pint for large passenger locomotives would seem to be the amount needed to lubricate properly. In bad water districts the oil allowance should be increased about 25%. The use of grease on crank pins and driving axles seems to offer the best solution of how to decrease the cost of external lubrication and at the same time secure better results. Grease gives results about as follows: Reduces engine failures due to heated journals and pins; reduces cost of lubrication; reduces cost of labor incident to inspection, cleaning and renewals of lubrication packing; reduces delays incident to oiling; reduces cut journals incident to oil lubrication; possibly produces a slight increase in machine friction.

The discussion of the report consisted entirely of statements of experience with grease lubrication from different members. In a word, these experiences bore out the statements made re-

CAST IRON FOR CYLINDERS.—The report of this committee recommended the following specifications for metal for locomotive cylinders:

Silicon	1.25—1.50%
Phosphorus	0.5 —0.8 %
Sulphur	0.06—0.1 %
Manganese	0.3 —0.6 %
Combined carbon	0.5 —0.7 %
Graphite carbon	2.75—3.25%

Tensile strength, 25,000 lbs. per sq. in; transverse strength, 3,000 lbs. on 1¼-in. round bar, 12 ins. between supports; deflection, 0.1-in. minimum on transverse test; shrinkage, ⅛-in. in 1 ft., maximum. For testing, the committee recommended the practice of the American Society for Testing Materials. The report received no discussion.

LOSS OF POWER FROM FRICTION.—A report of a series of tests made for the New York Central & Hudson River R. R. by Prof. W. F. M. Goss, of Purdue University, to determine the loss of power resulting from friction in the machinery of a modern passenger locomotive, was presented to the association by Mr. J. F. Deems (N. Y. C. &

R. R). Described briefly, the locomotive used ...the front end tests at Purdue was fitted up ...for grease lubrication and then for oil lubri... and comparative results obtained. The ...t of the comparison was: Accepting the oil ...brication as the basis of comparison, it appears ...at at 20 miles an hour the loss of power re...lting from the use of grease is slight—so small, ...fact, as to be almost negligible—but as the ...eed is increased the loss is increased, and at 60 ...iles per hour it amounts to from 140 to 160 HP. ...he equivalent coal loss, assuming 4 lbs. of coal ...r HP.-hour, is something more than 500 lbs. ...r hour.

This report brought the advocates of grease ...brication out in full force, and figures and ...xperience were cited to prove the value of ...rease lubrication. None of the data presented, ...owever, applied to the question at point, namely, ...s effect on the internal friction of the locomo...tive.

OTHER REPORTS.—The report of the Com...mittee on Locomotive Tests of the Pennsylvania ...R. R. at the St. Louis Exposition gave the re...sults of the tests as already published in our issue of Feb. 15, 1906. There was no discussion on the report proper. The report on Water Softening for Locomotive Use gave a series of statements of experience of three or four railways using systems of water softening. Neither the report nor the discussion, which turned onto the use of a boiler compound, was an adequate or useful treatment of the important subject. The Committee on Classification of Locomotive Repairs presented in its report a classification distinguishing (1) between those engines that are in service and those which are under repair; (2) between those engines which are under running repairs and those which are under shop repairs; and (3) between the various classes of shop repairs. Briefly, engines should be considered as under repairs if not ready for service within 24 hours of their time of arrival at the roundhouse; running repairs are those, whether made in shop or roundhouse, in which the estimated cost does not exceed $100; shop work is divided into machinery repairs, flue repairs and fire-box repairs. In the discussion several members disputed the value of such a classification of repairs, for the reason that while it showed the shop output it did not indicate the quality and thoroughness of the work done. The Committee on Mechanical Stokers for Locomotives reported that nothing new had come up in the line or use of such devices since its previous report. A paper on "The Modern Locomotive Injector," by S. L. Kneass, and one on "Welding Locomotive Frames," by R. P. C. Sanderson, were received and ordered printed, without discussion. Committee reports were also presented on Electricity on Steam Railroads and on Engine House Running Repair Work on Locomotives.

ELECTION OF OFFICERS.

The following officers were elected to serve the Association for the ensuing year: President, J. F. Deems, New York Central & Hudson River R. R.; First Vice-President, Wm. McIntosh, Central R. R. of New Jersey; Second Vice-President, H. H. Vaughan, Canadian Pacific R. R.; Third Vice-President, G. W. Wilden, Erie R. R.; Treasurer, Angus Sinclair.

THE RELATIVE CORROSION OF WROUGHT-IRON AND STEEL.*

By Henry M. Howe.†

On one hand we have the very general public opinion that steel corrodes not only faster but very much faster than wrought-iron, an opinion held so widely and so strongly that it cannot be ignored. Smoke does not prove that fire exists; but such strong smoke bids us look carefully for fire. On the other hand we have the results of direct experiments by a great many observers, in different countries and under widely differing conditions; and these results certainly tend to show that this popular belief is completely wrong, and that on the whole there is no very great difference between the corrosion of steel and wrought-iron. Under certain sets of conditions steel seems to rust a little faster than wrought-iron, and under others wrought-iron seems to rust a little faster than steel. Thus taking the tests in unconfined sea water as a whole, wrought-iron does constantly a little better than steel; and its advantage seems to be still greater in the case of boiling sea water. In the few tests in alkaline water wrought-iron seems to have the advantage over steel, whereas in acidulated water steel seems to rust more slowly than wrought-iron.

We, as technical and scientific men, naturally attach greater weight to the numerical results of careful direct comparative tests than to rumor and popular belief. When ultra conservative engineers used to cry out against steel boilers, it seemed to us like the old cry "Great is Diana of the Ephesians." Those who raised the cry, cried what they firmly believed was true, and what they thought every sensible man knew to be true. But they were wrong; and I need not tell this society that such cries and popular beliefs, even when widely and firmly held, often prove wrong. Compared with the results of direct comparative tests such beliefs have the great disadvantage of lacking all precise and definite foundation; they are easily spread from man to man. The fact that steel has come into wide use simultaneously with a great increase in the sulphurous acid in our city air and of strong electric currents in our city ground, may well lead the practical man, be he hasty or cautious, into inferring that the rapid corrosion of to-day is certainly due to the new material of to-day, steel, whereas in fact it may be wholly due to the new conditions of to-day, sulphurous acid and electrolysis. At the same time, while popular belief has the disadvantage of lacking direct numerical comparative data, it has the very great advantage of being based on the actual conditions of use, more closely, and often far more closely, than our direct comparative tests are, unless these are planned with very great care.

In view of this great discrepancy between popular belief and the results of our direct tests, it behooves us who have relied chiefly on these latter to examine their conditions carefully, to see whether they really have represented fairly the conditions of actual industrial use and service, in such a way that, if there is a real difference between the corrosion of steel and that of wrought-iron, such a difference would become manifest. In short, have our direct comparative tests been trustworthy?

Now, when I review our conditions in the light of this apparent contradiction between popular belief and the results of the differences between steel and wrought-iron which ought to cause a difference in their rapidity of rusting, there are three prominent differences: (1) Blowholes; (2) Manganese; and (3) The presence of cementite in the steel and of cinder in the wrought-iron. Let us take these up in order and see how they require that direct tests should be very prolonged or pushed to destruction.

(1) Blowholes exist in steel but not in wrought-iron. But blowholes, at least blowholes which do not weld up and thus cease to exist, are not necessary. Yet they are to be prevented only by care and skill. Hence, get your steel only from careful and trustworthy makers.

(2) Steel always and almost necessarily contains more manganese than wrought-iron. This may or may not hasten its rusting. If it does, then its effects ought to be made manifest even in short-time tests. From the fact that such tests do not show that steel rusts materially faster than wrought-iron, I infer that this manganese is probably not a serious cause of rusting.

(3) Steel is generally richer than wrought-iron in cementite, the iron carbide Fe_3C. Wrought-iron always contains very much more cinder than steel. Each of these substances, the cementite of the steel and the cinder of the wrought-iron, may have a double influence on corrosion, hastening it through difference of potential and retarding it by acting as a mechanical barrier like so much paint, to exclude the oxygen or the air or the water. It is not clear that the influence of difference of potential ought to change materially as corrosion proceeds; but it is clear that that mechanical protection given by the plates of cementite and of cinder ought to increase as corrosion proceeds. When a piece of wrought-iron, for instance, is first exposed to corrosion, only the outcrops, so to speak, of the sheets of cinder come to the surface; its mechanical protection is very small. But as corrosion proceeds, and more and more of the metal which at first overlay the sheets of cinder is eaten away, the remaining cinder forms a larger and larger proportion of the outer surface, and therefore protects a constantly increasing proportion of the underlying metal from corrosion. In short, the mechanical protection afforded by the cinder ought to increase as corrosion proceeds. Here, then, is a cause which, as corrosion proceeds, should continuously tend to retard the corrosion of wrought-iron, and to make it compare more and more favorably with steel. But, in like manner, as steel is gradually corroded away, more and more of its surface should come to be composed of cementite, and this fact should tend to retard the corrosion of steel, because cementite, too, should protect the underlying free iron or ferrite.

These causes may, in time, reverse the initial relative rapidity of rusting of steel and wrought-iron. Steel which in the first few months may rust faster than wrought-iron may, on greatly prolonging the experiments, or pushing them to destruction, actually rust more slowly, and vice versa.

Now of the two, the cinder of wrought-iron ought to gain more than the cementite of the steel, in its value as a mechanical retarder of corrosion, as time goes on and more and more of the metal is eaten away. The reason for this is that the cementite is in such extremely minute microscopic plate that the eating away of a very small quantity of the iron from above them ought to bring very nearly the full proportion of this cementite to the surface; whereas the much larger and more distantly scattered plates of cinder in wrought-iron would not constitute their full share of the surface until a much thicker layer of initially overlying metal had been eaten away. This, then, may be the true explanation: that is, the reason why steel does not rust faster than wrought-iron in our direct tests, though it does in actual use, is that our direct tests are too short to bring out the full protective action of the cinder of the wrought-iron. Or the reverse may be true. As time goes on, the harmful effect of the difference of potential of the cinder may grow more than its protective action. Let us, therefore, henceforth push our tests to destruction.

Two other points: Sheet steel roofing may rust faster than iron because it holds the paint better, and yet steel in other forms, like tubing, may rust no faster than wrought-iron. Again let me emphasize the difference between different steels. Carelessly made steel, containing blowholes, may rust faster than wrought-iron, yet carefully made steel, free from blowholes, may rust more slowly. Recognize that any difference between the two may be due not to the inherent and intrinsic nature of the material, but to defects to which it is subject if carelessly made. Care in manufacture and special steps to lessen the tendency to rust, might well make steel less corrodible than wrought-iron, even if steel carelessly made should really prove more corrodible than wrought-iron.

*A paper read before the American Society for Testing Materials, at Atlantic City, N. J., June 22, 1906.

†Professor of Metallurgy, Columbia University, New York, N. Y.

THE RICHMOND & CHESAPEAKE BAY RAILWAY CO. has been incorporated to build a single-phase electric road between Richmond and Chesapeake, Va. About 15 miles of track, the portion lying between Richmond and Ashland, will be equipped immediately, and will parallel the Richmond, Fredericksburg & Potomac R. R.

In general the construction of this line will correspond to previously constructed single-phase roads, except that the trolley voltage will be higher. The catenary method of suspension will be used, adapted for a trolley potential of 6,600 volts. Each of the cars will be equipped with four G. E. A.-603 (125) AC single-phase motors. Multiple unit control will be furnished using the Sprague-General Electric system. The air-brake system will be of the combined straight and automatic type with C. P. A.-52 motor compressors.

Power for the operation of the new road will be furnished by the Virginia Passenger & Power Co., and the arrangements for furnishing the single-phase current are of especial interest. Two generating sets will be furnished, both to be operated ordinarily by water power, but arranged for electrical drive when that is necessary. The first unit will consist of a 750-KW., 6,600-volt, 3-phase, 25-cycle, generator, mounted on the same shaft with a 750-KW., 2,300-volt, 60-cycle, 3-phase generator, the shaft being extended at one end for connection with a water-wheel of sufficient power to drive both generators at their rated output. The second set will be made up of a 25-cycle, 6,600-volt generator, a duplicate of the first, mounted on the same shaft with a 750-KW., 550-volt direct current machine. When there is sufficient water to operate all of the water-wheel generators in the station, these sets will be driven by the water-wheels. If at any time there is not sufficient water to drive the generators, or flood conditions render it advisable to close the gate valves, the sets will be disconnected from the wheels and operated as straight motor generator units, the 60-cycle generator being run as a 60-cycle synchronous motor and the 550-volt machine being operated as a direct current motor. Power for driving the motor end of the 25-cycle sets, in this case, will be obtained from the present engine driven units of the company. When the motor generator units are operated as water-wheel driven machines, the 60-cycle generator will be operated in parallel with the present 60-cycle machines, and the 550-volt machine of the other set will run in multiple with the present 550-volt machines. The 750-KW., 25-cycle generators in each set will for the present be operated as single-phase machines and will supply current directly to the trolley line of the Richmond & Chesapeake road at 6,600 volts.

In addition to the above apparatus there will be a lighting station at Ashland. This will consist of a 100-KW., 2,300-volt, 60-cycle, 3-phase generator mounted upon a common base with a 150-HP., 440-volt, 25-cycle, single-phase induction motor. This set will be operated from the 6,600-volt trolley, through a 150-KW., single-phase, oil-cooled transformer. The motor generator set will be provided with a direct connected exciter mounted on an extension of the shaft. The voltage will be controlled by a Tirrill automatic regulator, made by the General Electric Co., which will insure a uniform voltage at the generator end of the lighting service regardless of the fluctuating trolley voltage.

TWO-HINGED ARCH TRUSSES FOR THE NEW LIVE-STOCK PAVILION, CHICAGO.

A large exhibition hall, or "Livestock Pavilion," has been built for the Union Stock Yard & Transit Co., of Chicago, for the purpose of the annual livestock exhibition held at the Chicago stock yards. Until last summer the exhibition was held in two parallel buildings, 130 × 600 ft., 40 ft. apart, one being for horses and the other for cattle; in the center of the former was an amphitheater, 130 ft. diameter, surmounted by a steel dome, and here the show ring was located. In order to enlarge the exhibition hall and the ring, a new steel building, 200 × 300 ft., has been erected transversely of the old buildings, the middle portion of these being torn down for a width of 200 ft., as shown by the sketch plan Fig. 1. The old dome is left intact, as are the seats at the end of the old amphitheater.

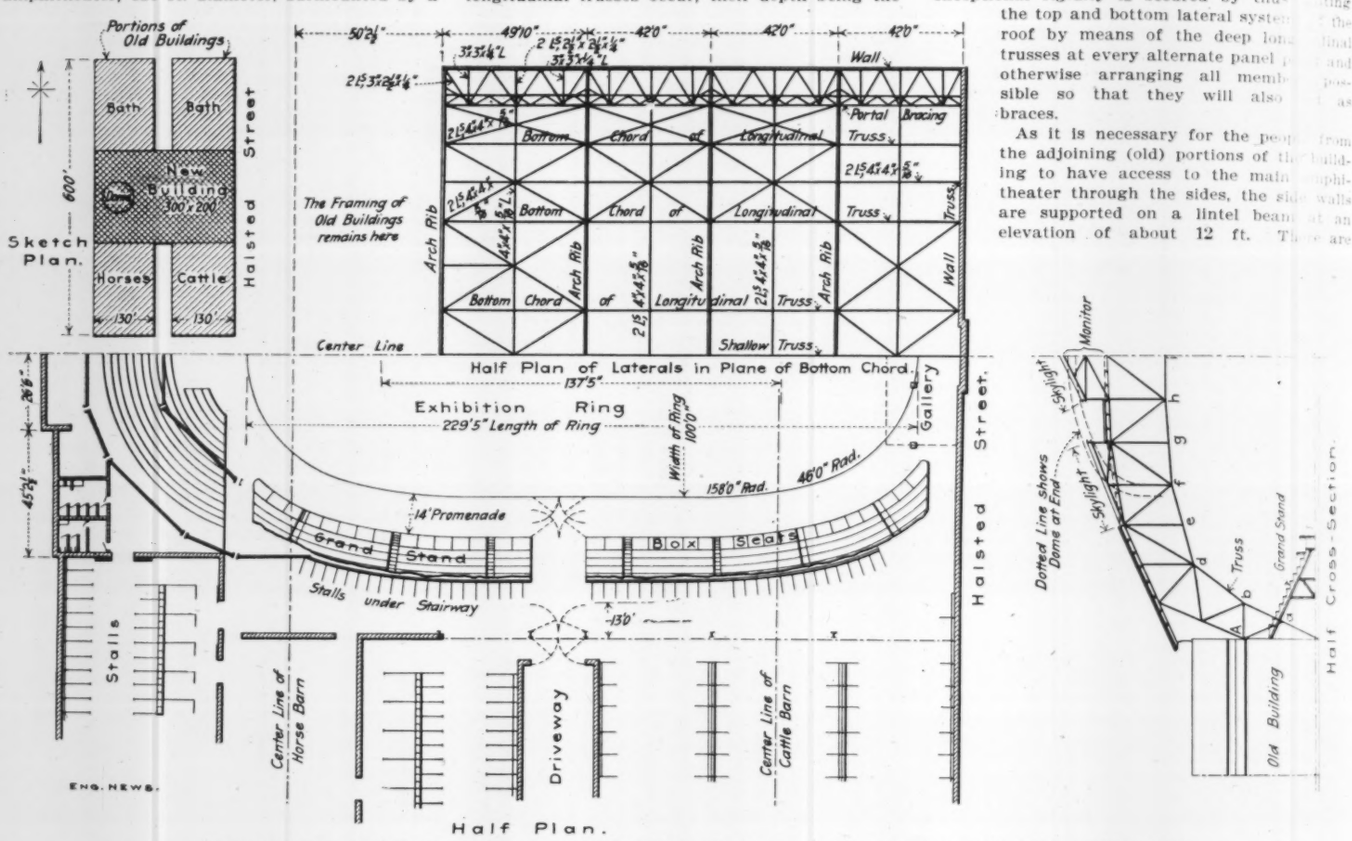


FIG. 1. SKETCH PLAN OF NEW LIVE-STOCK PAVILION; UNION STOCK YARD & TRANSIT CO., CHICAGO.

The principal structural feature is the use of two-hinged arch trusses, there being no columns in the main part of the building. These trusses are of 200-ft. span, with a height of 53 ft. from the top of the foundation masonry to the bottom chord; the depth of truss is 20 ft. at the center, and a monitor roof makes the total height about 90 ft. The thrust, which amounts to about 56 tons, is taken by tie-rods connecting the feet of the trusses, these tie-rods being embedded in concrete beneath the floor. The general design of these trusses is shown in Fig. 2. The chords are composed of two 12-in. channels of varying section and weight, reinforced by 12-in. side plates where necessary and connected by lacing bars over the flanges. The web members are each composed of four angles, either connected by lacing or with a plate. The first truss supports the dome, Fig. 3, which represents a load of 40 tons; this was formerly carried by columns, which have now been removed. The roof load which these trusses were figured to carry (in addition to the dead weight of the material in the roof) was as follows: (1) A superimposed load of 25 lbs. per sq. ft. of horizontal projection; (2) a wind pressure (assumed to be blowing horizontally) of 30 lbs. per sq. ft. of vertical projection, and (3) a snow load of 10 lbs. per sq. ft. of horizontal projection. The complete truss weighs 40 tons, and is proportioned to carry 280 tons.

The end wall truss and end framing are shown in Fig. 4. The arch trusses are spaced about 42 ft. c. to c., and between them are longitudinal trusses framing into the arch trusses at the panel points of the latter, which are about 16 ft. c. to c. The top and bottom chords of the main arch trusses break on alternate panel points. At the points at which the lower chords break deep longitudinal trusses occur, their depth being the same as that of the arch trusses. This method of framing adds considerable stiffness to the bottom chords of the arch trusses, and at the points where it is most needed. The alternate longitudinal trusses (those which occur at the panel points at which the top chord is spliced) are but 7 ft. deep, as they are only depended upon to sustain their portion of the roof load, and incidentally serve as struts for the top chord bracing. The longitudinal trusses support 8-in. channel rafters, which extend transversely of the building, and these rafters carry 5-in. channel purlins about 5 ft. 4 ins. c. to c. The roof sheathing is of 3-in. matched and dressed plank, which rests directly on the purlins, and is covered with fireproof composition roofing.

The top lateral bracing occurs in all [illegible] the plane of the tops of the top chord [illegible] bottom flanges of the roof rafters, and [illegible] to the latter at the intersection points. [illegible]tom chord bracing occurs in all panel[illegible] two end bays, and in the panels of [illegible] mediate bays, where the bottom chor[illegible] arch trusses is in compression, due to [illegible] effect. The bottom chords of the de[illegible]tudinal trusses before mentioned form [illegible] of the bottom lateral bracing. It is clai[illegible] exceptional rigidity is secured by thus [illegible] the top and bottom lateral syste[illegible] roof by means of the deep lon[illegible] trusses at every alternate panel [illegible] otherwise arranging all memb[illegible] sible so that they will also [illegible] braces.

As it is necessary for the peo[illegible] the adjoining (old) portions of th[illegible] ing to have access to the main [illegible] theater through the sides, the sid[illegible] are supported on a lintel beam [illegible] elevation of about 12 ft. Th[illegible]

The two wall columns between each of the main arch trusses, thus making three sub-panels of about 14 ft. each. Below the beams and between the tops of the columns a system of portal bracing was introduced, and this, with the continuity of the columns (which extend to the top of the side walls), adds to the rigidity of the sides. In one of the main panels this portal bracing was omitted, to be replaced by cross bracing connecting the tops of the columns with the anchorage at the base. The building is lighted by windows about 15 ft. in height, just under the eaves; by skylights in two panels on each side of the main roof, and by the monitor on the center line of the building, with windows 7 ft. high in its sides. This provision is said to be ample.

DIMENSIONS OF NOTABLE LARGE SPAN ROOFS.

	Chicago L. St. Pav.	Chicago Coliseum. (New)	Chicago Coliseum. (Old)	St. Louis Coliseum.	Kans. City Coliseum.
Roof truss	2-hin. arch.	3-hin. arch.	3-hin. arch.	3-hin. arch.*	Triangular
Span	198 ft.	149 ft. 9 ins.	215 ft. 0 in.	178 ft. 6 ins.	187 ft. 4 ins.
Rise	54 "	66 ft. 0 ins.	73 " 0 "	80 " 0 "	.....
Max. depth of truss	25 "	15 " 7 "	25 " 0 "	.....	32 ft. 6 ins.
Truss spacing	42 "	22½ to 25 ft.	46 " 8 "	36 ft. 8 ins.	23 " 6 "
Hall	200 x 300 ft.	160 x 300 ft.	200 x 600 ft.	120 x 220 ft.	180 x 300 ft.
Seating cap	10,000 ft.	.....	.....	7,000	25,000
In Eng. News	This issue	Sept. 14, '99	Nov. 12, '96	Aug. 10, '97	{Nov. 2, '99 / July 5, '00

* The St. Louis trusses had no tie-rods between their feet.

Building.	Roof truss.	Span.	Rise.	In Eng. News
Manufactures Bldg, Chicago Ex	3-hin. arch.	368 ft. 0 ins.	206 ft. 4 ins.	Sept. 1, '92
U. S. Govt. Bldg., St. Louis Ex	"	172 " 0 "	66 " 9½ "	" 29, '04
69th Regt. Armory, New York	"	189 " 8 "	103 " 4 "	June 1, '05
Trainsheds:				
Penn. R. R., Jersey City	"	252 " 8 "	89 " 9¾ "	{Dec. 27, '00 / Sept. 20, '01
" " Philadelphia	"	259 " 0 "	88 " 3¾ "	Jan. 15, '03
Phil. & R. R. R., Philadelphia	"	300 " 8 "	100 " 4 "	June 1, '03
Prussian Rys., Cologne	2-hin. arch.	209 " 6 "	79 " 0 "	Oct. 6, '92

The grand-stand (Fig. 5) is composed of 12-in. I-beam stringers, with risers and treads of 2 × 2-in. angles laced together and covered with 2 ins. of expanded metal and concrete. The lacing is of angles 2 × 1½ ins., with the legs set back so as to leave a plane surface for the attachment of the expanded metal directly to the face of the steel. The steel in the grand-stand is self-sustaining, and the risers are figured to take the entire load, no dependence being placed on the concrete, which is considered as a veneer only.

The total weight of the steel in the building is 2,850 tons, and it was all erected in less than five weeks. The building was opened for the Live-Stock Exhibition in December last. The main dimensions are as follows: Size of building, 600 × 300 ft.; size of auditorium, 300 × 200 ft.; size of arena, 230 × 100 ft.; capacity of auditorium, 10,000 people; total floor space, 243,600 sq. ft.; cost, $300,000.

The architects were Frost & Granger, of Chicago, and the general contract was awarded to the Wm. Grace Co., of Chicago. The steel work was designed and manufactured by the Modern Steel Structural Co., of Waukesha, Wis.

In connection with the description of this building, we give in the accompanying table the principal details of the roofs of other large buildings of somewhat similar design, which have been described in our columns, as well as the dimensions of some noted train-sheds, etc., having arched trusses. It may be noted here that of the two Coliseums in Chicago both collapsed during erection and were rebuilt; the first one, however (at 63d St.), was destroyed by fire in December, 1897. The great Manufacturers Building at the Chicago Exhibition of 1893 was also destroyed by fire.

PACIFIC COAST TIDES AND THE DETERMINATION OF MEAN SEA LEVEL.*

By W. B. Dawson,† M. Inst. C. E.

In any tidal observations, the two essential objects are the correct times of the various tide stages and a plane of reference for heights—Mean Sea Level. The main object of the Tidal and Current Survey, as a branch of the Marine Department, is to deal with the time of the tide, since this is the matter of chief importance to navigation and the question of levels is quite secondary for that interest. But the value of reliable levels, which can only be obtained from tidal observations, makes it seem right to take the additional trouble necessary to secure them. These levels are of direct value in the construction of wharves, dredging and other harbor improvements, and in city work. Mean Sea Level, the most important datum obtained from tidal records, has a much more general importance. To understand the best method of obtaining its location, it is necessary to explain briefly the character of the Pacific tides, as at first sight they appear quite irregular.

*Extract from a report entitled "Tide Levels and Datum Planes on the Pacific Coast of Canada," forming Supplement No. 1 to the 38th Annual Report of the Department of Marine and Fisheries of the Dominion of Canada; also presented before the Canadian Society of Civil Engineers, February, 1906.

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In all parts of the world, the tides are found to accord with the varying movements and distances of the moon and the sun. In the North Atlantic it happens that they are chiefly influenced by the moon's phases. It was thus supposed that the primary characteristic of all tides was a marked alternation in height from springs to neaps in the period of the synodic month. The tide of the Pacific, however, can best be described as a declination tide. Its leading feature is a pronounced diurnal inequality in time and height, which accords with the declination of the moon; and this is also subject to an annual variation with the change in the declination of the sun. The period in which the diurnal inequality recurs is the tropical or declination-month, of 27.2 days; which is shorter than the synodic month and gradually falls back through its period in successive months. As the solar influence is unusually large in the Pacific relatively to the lunar, the annual variation is the more accentuated.

On the open coast of the Pacific, the tide curve is still fairly regular, though showing the diurnal inequality strongly; and in some regions, especially northward, the springs and neaps can be distinguished with little difficulty. But in the Strait of Fuca and the region of the Strait of Georgia, which makes up half the coast line of British Columbia and where all the more important harbors are situated, the appearance of the tide curve is anomalous. The high waters are nearly at the same level; and the range depends on the amount of fall to low water, which may be almost inappreciable or very pronounced. During the greater part of the day, there may thus be a long stand or only a slight fluctuation near the high-water level; with a sharp and short drop to the lower low water which occurs once in the day. This type only changes to a fairly symmetrical curve when the moon is on the equator near the time of the equinoxes.

The spring and neap tides are thus reduced to a secondary feature which is usually obscured by the stronger characteristics of the tide. The "Establishment," which is so well marked in the Atlantic, is here almost illusory; unless it is strictly reduced to equinoxal and equatorial conditions, in accordance with the definition used in France. In dealing with tide levels, it may still be convenient to speak of spring and neap tides, if they are understood to mean the two maxima and the two minima in range or in level which always occur in the period of the lunar month. But the two highest and the two lowest points on the tide curve for the month may be as much as five days before or after the full or new moon, as they are so largely occasioned by the diurnal inequality, dependent on declination.

The extreme tides of the year necessarily occur at the nearest point to the solstices at which the moon reaches its maximum declination.

A tide of this character is apt to be termed irregular by the mariner; as the tropical or declination-month, which is its governing period, is less familiar and less noticeable than the synodic month of the moon's phases. It is evident, however, that this tide is perfectly astronomical; and when reduced by harmonic analysis its prediction is just as definite as for any other type of tide.

With a tide of this type, there is a notable difference between the half-tide level and the true value of Mean Sea Level. Its only accurate value is the mean ordinate

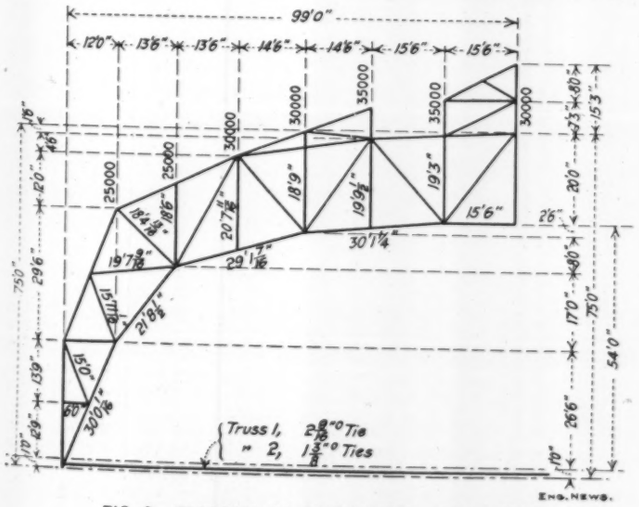

FIG. 2. DIAGRAM OF TWO-HINGED ARCH TRUSS.

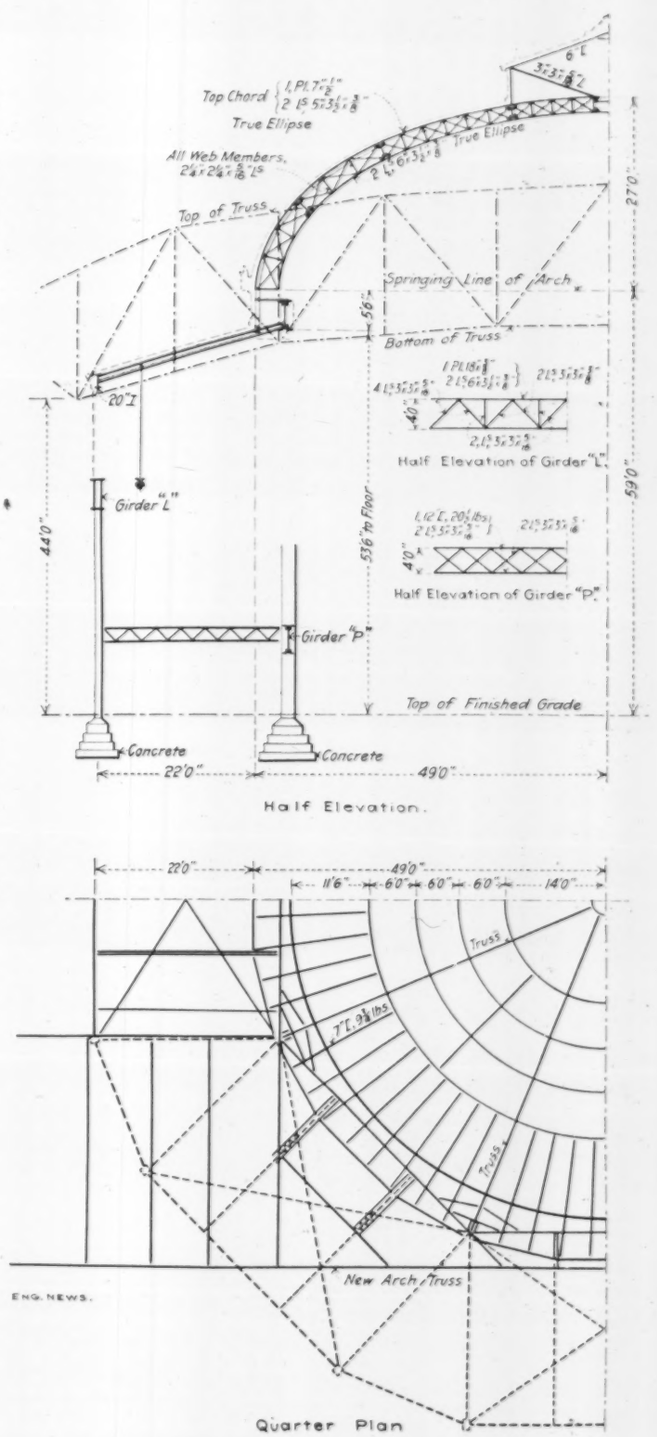

FIG. 3. STEEL DOME, SUPPORTED BY ARCH TRUSS.

found by the integration of the tide curve, referred to any invariable base line or datum. This mean ordinate fixes the position of the horizontal line which bisects the area of the tide curve; and this also accords with the best definition of Mean Sea Level for any type of tide. The half-tide level may differ a whole foot from true Mean Sea Level, even in the case of a tide whose extreme range is only 13 ft.

The advantage of a registering tide gage is much emphasized with tides of this character. If scale readings are taken by direct observation, which the Admiralty surveyors usually prefer, they must be continuous, day and night, and afterward plotted as a curve; or little use can be made of them except for the reduction of soundings.. With a registering gage, this elaborate and expensive method can be dispensed with. The hourly ordinates of the tide curve throughout the year enable the true value of Mean Sea Level to be readily found; and even with a shorter period, the continuity of the record enables the diurnal inequality to be followed; and if this is known, the average level and the extremes of high and low water, and other data, can be correctly determined. The continuous record is equally important with respect to the time of the tide, in which there is a similar inequality of interval; but with this we are not now dealing.

The question of Mean Sea Level is of unusual interest on the Pacific Coast, as there is reason to believe that its elevation is changing. Some indications point to a rise in the level of the coast, at as high a rate as one or two feet per century. It is only from tidal observations properly reduced that any trustworthy result can be arrived at; and if the change is as rapid as supposed, it will not require an interval of many years to obtain a fair approximation to its amount.

AN ELECTRIC HAULAGE SYSTEM FOR MINES, which is being used in a number of coal mines, is that in which the conductor is a rack-rail from which the current is taken by a cogwheel on the locomotive (Engineering News, March 17, 1904). The rack is a steel bar or plate with transverse slots, and its edges are fitted to grooves in longitudinal timbers so as to leave only a narrow slot for the cogwheel. In the mines of the Donahoe Coke Co., at Greenwald, Pa., with a daily output of about 2,000 tons, there are five main entries, parallel with each other; the middle one and the two outer ones are air ways, while the two intermediate entries are haulage ways; the grades in these latter were originally from 1% to 9%, but the irregular profile has been improved and the grades are now only 1.8 to 3.8%; the length of each main haulage way is about 3,000 ft., and the mine has about 10,000 ft. of electric haulage. At intervals of 400 ft. there are face entries, approximately at right angles to the main entries, and these are made level in order to facilitate the work of collecting the loaded cars from the headings by horses. The cars thus collected are put together, and when a sufficient number has been collected to form a train for the locomotive the attendant switches on an electric light at the junction of the face entry with the main entry, so as to notify the motorman. When the face entries become too long for the convenient or economical use of horses, the haulage system is extended into them, using for this purpose the worked out sections from the main entries. There are two electric locomotives, of 100 HP. and 160 HP., and these have handled as much as 2,200 to 2,500 tons per day, most of the work being done by the 100 HP. engine. On March 15, the latter engine made 35 trips of 4,600 to 7,900 ft. each, and hauling 828 cars in all. The total distance traveled was about 40 miles, the average length of haul being 6,025 ft., and the average time for the round trip being 16 minutes. The loaded cars average 7,500 lbs. each (4,500 lbs. of coal), and the total weight of a 25-car train is about 93½ tons. The amount of coal hauled was about 1,863 net tons. The equipment was furnished by the Goodman Mfg. Co., of Chicago.

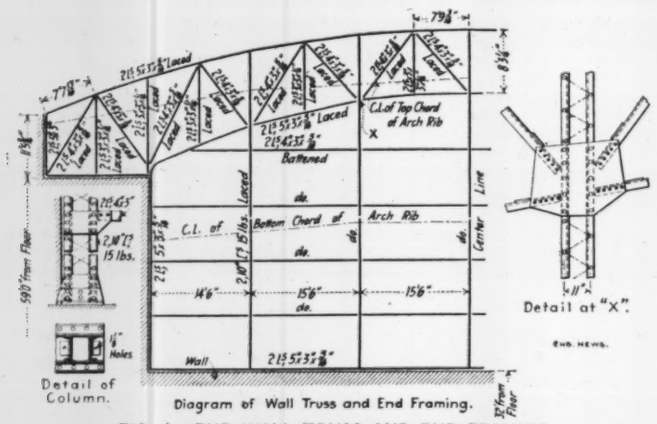

FIG. 4. END WALL TRUSS AND END FRAMING.

EMPIRICAL FORMULAS FOR REINFORCED ARCHES.

By Daniel B. Luten.*

Reinforced concrete arches are proving deservedly popular on account of their durability, handsome appearance and economy, as compared with other bridge structures, but their efficient design remains an enigma to the busy engineer. That this will not remain a serious obstacle to their adoption is proved by the fact that steel bridges which are much more complicated in erection and whose design is not thoroughly understood except by trained experts, are in extended and successful use, and that, too, when the possibilities of errors in erection of steel bridges are many times greater than in the simple concrete structure.

In the following discussion a method of empirical design is arrived at that will enable a designer to lay off all the details of the section for an 80 ft. span in less than an hour, yet with great accuracy and efficiency. The method has been applied to upwards of 100 arches of spans from 5 ft. to 100 ft., no one of which has shown any signs of weakness. Such results cannot, however, be attained by empirical methods with arches subject to unequal abutment reactions and with concentrated systems of reinforcement. Several tests with a small wooden model will illustrate some of the weaknesses of the ordinary voussoir arch and also point the remedy in the more efficient reinforced arch.

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The model employed was a small arch of hardwood blocks, nine in number, fitted to form an oval opening of 8 ins. span by 2 ins. high, with crown ½ in. thick and base of abutments 2 ins. The model was 2½ ins. wide with grooves cut in each face to receive a steel wire reinforcement. The joints between blocks were normal to the intrados, and each joint was fitted with dowel pins 1/32 in. long to hold the blocks in alinement. The model is shown to scale in Fig. 1, in elevation and plan. The arch abutments are loosely notched into a removable base, in order to illustrate the importance of abutment support against the horizontal thrust of the arch.

The grooves for the reinforcing wires are continued under this base to anchorage. The model as designed, without the reinforcement or timber base, represents a reasonable design for a small stone arch except that no provision is made for the loading of earth fill or the resistance of the banks to the horizontal thrust of the arch.

Fig. 2 shows the model erected on a hard, smooth surface without the base board and no reinforcement. It is barely self-supporting and collapses under a weight of eight ounces, as shown in Fig. 3.

The abutments have been forced back by the thrust of the arch, and it is apparent that unless they can be assisted in some way to furnish an adequate support for the arch, no great strength can be developed. The customary method of increasing the resistance of the abutments has been to add masonry, but to resist horizontal forces by masses of masonry is an exceedingly inefficient device as compared with a tension member from abutment to abutment when that is feasible.

This tension member is introduced in Fig. 4 by the base board which is the only modification from Fig. 2; yet the model now supports 3 lbs. safely, or more than six times its former load. We have thus increased the efficiency of the model six times by introducing the abutment tie. In this form, the model fails, however, under a load of 4 lbs. by rupture of the arch ring. The abutments have held securely with the assistance of the base board, but the joints in the arch have opened at the inner surface along the crown and at the outer surface along the haunches. Failure has been caused directly by the rupture of the joints that are numbered 2, 3, 5 and 6, of Fig. 1. If these joints could be made to resist rupture, this new source of failure would be eliminated.

By embedding a steel tension member so as to cross the joints of rupture as near as possible to the surface, their tendency to open may be resisted to the full strength of the steel. No joint can open at both edges at the same time. Consequently a single reinforcing member may be passed continuously through all the joints at their edges of rupture. The model when loaded at the crown shows very plainly that the joints tend to open at 1, 2, 3, 4 along the crown at the inner surface of the arch, Fig. 1, and at 5, 6, 7 and 8 along the outer surface at the haunches. The reinforcement should then follow the heavy line of Fig. 1, crossing the arch ring between 1 and 5 and between 4 and 7. In the model a groove about 1/16 in. deep is chiseled in each face to follow this line permitting the insertion of a small wire which can be anchored at each end in the base board.

The objection can, however, be made that such a reinforcement is not of practicable application since it provides only for a load at the crown. But in an actual structure with the tension mem-

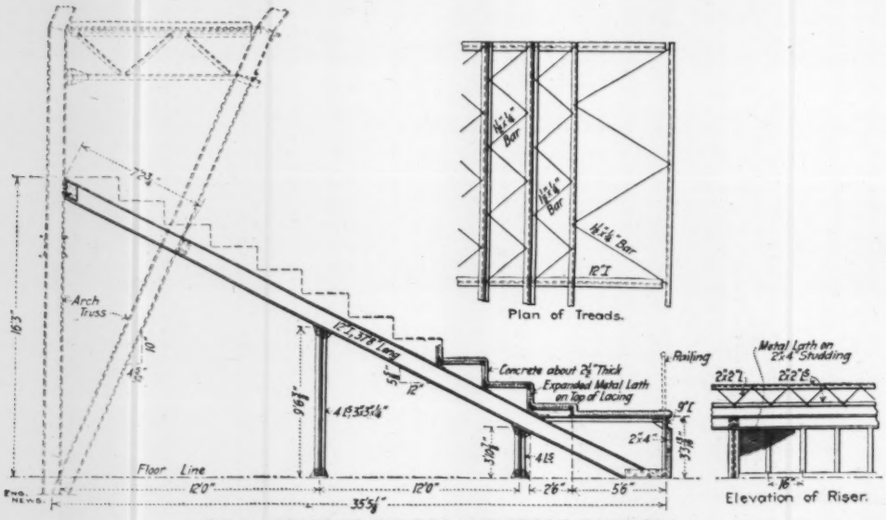


FIG 5. GRAND STAND FOR LIVE-STOCK PAVILION.

...ced at intervals of a foot say, alternate ... can be made to cross the arch ring at ... distances from the crown, as shown by ... lines in Fig. 1; and it can be shown ... distributing the points of crossing over ... middle one-third of the half arch, all possible tensile and shearing stresses will be provided for, for any possible live load on an earth covered arch.

In Fig. 6 the model is shown again, this time reinforced with a No. 20 wire (an ordinary broom wire) in the groove in each face, and now supporting 100 lbs. The model thus reinforced collapsed under a load of 160 lbs. by the breaking of the wires at one haunch. The reinforcement was the only modification from Fig. 4, yet the arch supports forty times as great a load, and 250 times the load carried by the model of Fig. 2, which had neither abutment ties nor arch reinforcement.

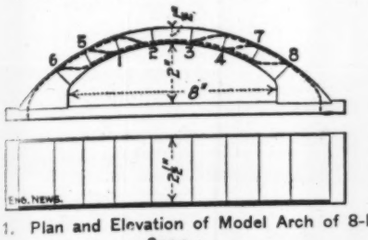
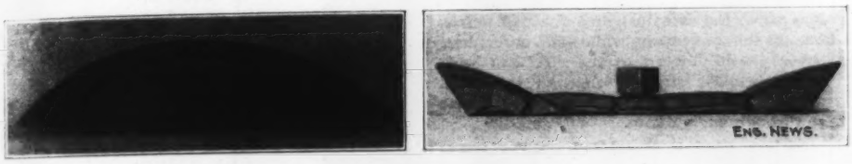
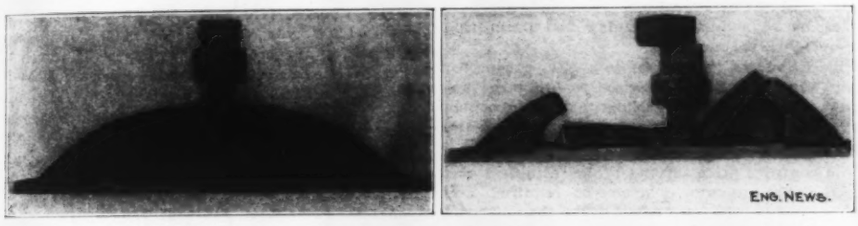


Fig. 1. Plan and Elevation of Model Arch of 8-In. Span.



FIGS. 2 AND 3. VIEW OF MODEL ARCH WITHOUT REINFORCEMENT SUPPORTING ITS OWN WEIGHT AND FALLING UNDER 8-OZ. LOAD.



FIGS. 4 AND 5. MODEL ARCH WITH ABUTMENTS HELD AGAINST SPREADING, BUT WITHOUT REINFORCEMENT, SUPPORTING 3-LB. LOAD AND FALLING UNDER 4-LB. LOAD.

We have evidently overcome some of the most serious ambiguities of the arch by these simple means. The abutments are now secure, the reactions vertical; and the reinforcement consists simply of a steel tension member in conjunction with a concrete compression member. In a model of this miniature form, it cannot be assumed that actual conditions of service can be duplicated. But in the actual working structure the addition of the earth filling over the arch to form the roadway increases the stability of the abutments and distributes the concentrated loads over the arch ring, reducing the severity of the test as applied to the model.

By applying these two features to a full-sized structure it becomes susceptible of much more rigid analytical examination than the ordinary type of arch, and its action under loads can be predicted with considerable accuracy. If the curves of the arch ring be so chosen as to follow the curve of equilibrium of the loading of earth and concrete, the efficiency of the structure is further increased and the analysis simplified. The reinforcement then need only resist the live load moments.

We know that an arch having for its inner curve a mean between the semi-ellipse and the segment of a circle, of the same span and rise, and for its outer curve a segment of a circle, is approximately in equilibrium under an earth fill. Assuming then that the arch is a linear arch of the above form, with abutments immovable and arch reinforced against rupture by a single tensional member, the following relations may be proved:

1. That in a linear arch the crown thickness varies directly as the thrust.
2. That the thrust varies directly as the span and as the load, and inversely as the rise.
3. That the fixed load varies directly as the span, and as the rise plus crown thickness plus fill over crown.
4. That the increase in crown thickness due to a concentrated load is approximately proportional to the bending moment at the crown divided by the square root of the crown thickness.
5. That the bending moment at the crown varies directly as the live load thrust, and as its eccentricity.
6. That the live load thrust varies directly as the live load, and that its eccentricity varies directly as the rise.

If, then, C_1 = crown thickness for fixed load only, C_2 = the added thickness for uniform live load, and C_3 = the added thickness for concentrated live load, the total crown thickness may be represented by $C = C_1 + C_2 + C_3 + 4$, where the

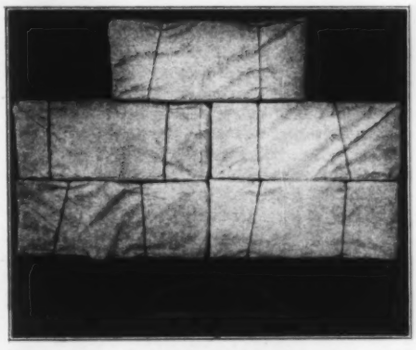


Fig. 6. Model Arch with Reinforcement Supporting 100-Lb. Load.

constant value 4 is added in order that for no span may the crown thickness fall below a reasonable practicable minimum.

Now, by the preceding propositions,

$$C_1 = a\frac{S^2}{R}(bR + C + F)$$

where S is span, R is rise, and F is fill over crown.

$$C_2 = c\frac{S^2 U}{R}$$

where U is uniform live load per square foot, and

$$C_3 = d\frac{L\,S}{R} + e\frac{L\,S\,R}{R\sqrt{C}}$$

where L is concentrated live load; a, b, c, d and e being constants. In the equation for C_1, C may be assumed to equal C_1, since for long spans the error is comparatively slight, while for all spans it is more than offset by the constant 4. In the equation for C_2, C may be assumed to be proportional to S^2 with slight error, compensated by the constant. The equation for crown thickness then becomes

$$C = \frac{a\,S^2(bR + F)}{R - a\,S^2} + \frac{c\,S^2\,U}{R} + \frac{L(dS + eR)}{R} + 4.$$

Thus far the crown formula has been deduced from theoretical considerations alone, and the constants a, b, c, d and e might be evaluated in a similar way, but only with great mathematical labor. But, with several hundred of these arches erected and in successful service, it is a more simple matter to adopt empirical methods by substituting in the formulas the dimensions of actual structures and solving the equations for the constants. By substituting for the variables their values from the dimensions of existing rational structures, a series of formulas will be obtained from which the values of the constants may be determined, five substitutions being sufficient to determine values of the five constants.

By making these substitutions and solving for the constants, and checking their values by additional solutions, the formula becomes,

$$C = \frac{3\,S^2(R + 3F)}{4{,}000R - S^2} + \frac{U\,S^2}{30{,}000R} + \frac{L(S + 5R)}{150R} + 4$$

in which C is crown thickness in inches, S is span in feet, R is rise in feet, F is fill over extrados in feet, U is uniform live loading in pounds per square foot, and L is live load in tons that will be concentrated on single track or single roadway over entire span.

The area of steel reinforcement required in the arch ring will be proportional to the bending moment at the crown divided by the crown thickness. The diameter of the rods then will be

$$A = \sqrt{D\frac{S\,L}{250C}},$$

where D is the distance between the rods in feet and A is the diameter of each rod in inches. The area of steel required to resist the horizontal thrust by steel ties from abutment to abutment is easily determined when it is remembered that the crown thickness is an approximate measure of the thrust, and that from this should be deducted the resistance of the embankment which may be assumed at about one ton per sq. ft.

Then $P = 1/5\sqrt{D(2C - H)}$,

where D is distance between rods in feet, P is diameter of rods in inches, and H is height of opening from crown to abutment ties in feet. It is for most locations a simple matter to tie the abutments with steel rods across the bed of the stream, and by embedding them in a thin pavement of concrete, to render them impervious to rust and at the same time make the bridge floodproof and provide a foundation for the falsework and centering. The allowable stresses for which the above constants were determined, are 300 lbs. per sq. in. for concrete in compression, and 15,000 lbs. per sq. in. for steel in tension.

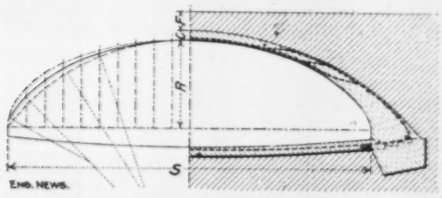


Fig. 7. Section of Arch to Illustrate Method of Design.

These formulas may be summarized and the method of design stated concisely as applied to the section of Fig. 7.

METHOD OF DESIGN.

1. TO DETERMINE CURVE OF INTRADOS.—Draw an ellipse of the required span and rise. Pass a segment of a circle through crown and springings of ellipse. Bisect the vertical ordinates between the ellipse and the circle. Approximate the resulting curve by arcs of cir-

cles, adjusting the curve at the springings to become tangent to the verticals.

2. TO DETERMINE THE CROWN THICKNESS.—

$$C = \frac{3\,S^3\,(R + 3F)}{4{,}000R - S^2} + \frac{U\,S^2}{30{,}000R} + \frac{L(S + 5R)}{150R} + 4.$$

3. TO DETERMINE CURVE OF EXTRADOS.—The curve of extrados is a segment of a circle of radius $E = I + \frac{C}{5}$. The extrados circle should be produced to its intersection with the abutment ties produced; but if such intersection does not occur within 75° of the crown of the extrados circle, then the back of abutment is determined by a tangent to this circle having a batter of one in four.

4. TO DETERMINE ABUTMENT TIES.—

$$P = \frac{1}{5}\sqrt{D\,(2C - H)}$$

5. TO DETERMINE ARCH-RODS.—

$$A = \sqrt{D\,\frac{8\,L}{250C}}$$

Where C is crown thickness in inches,
S is span in feet,
R is rise from springing to crown in feet,
F is fill over crown of extrados in feet,
H is height of opening from pavement to crown in feet,
U is uniform live loading in pounds per square foot,
L is moving load that will be concentrated on single track or single roadway, over entire span, in tons, of 2,000 lbs.,
E is radius of extrados in feet,
I is radius of intrados at crown in feet,
P is diameter of abutment tie in inches,
A is diameter of arch-rod in inches,
D is distance between rods in feet.

APPLICATION TO 80-FT. SPAN.

It was required to design an arch of 80-ft. span with 12 ft. rise and 16 ft. roadway across Clifty

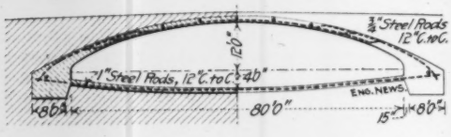


Fig. 8. Section of Arch Over Clifty Creek.

Creek, about six miles north of Greensburg, Ind. The mean depth of water was 3 ft., and the elevation of the proposed roadway about 18 ft. above the bed of the stream. The cross-section is shown in Fig. 8, in which the inner curve is a five centered curve bisecting the vertical ordinates between an ellipse and a segment of a circle, and having a radius of 88 ft. at the crown. The loading specified for this bridge was 200 lbs. per sq. ft. of uniform live loading besides a concentrated load of one 20-ton road roller. The fill over the crown was to be 1 ft. in depth. Substituting in the formula for crown thickness,

$$C = \frac{3x80x80\,(12+3x1)}{4{,}000x12 - 80x80} + \frac{200x80x80}{30{,}000x12} + \frac{20(80+5x12)}{150x12} + 4 = 16 \text{ ins.}$$

Laying off this thickness at the crown, the extrados circle is drawn with a radius $E = 88 + \frac{16}{5} = 91$ ft. 3 in. and extending to the ends of the abutment ties.

Assuming the spacing of the abutment ties as 12 ins., the diameter of rods required is,

$$P = 1/5\sqrt{1\,(2 \times 16 - 16)} = 4/5 \text{ ins.}$$

As these rods were, however, required to be welded in the field from 16-ft. stock, the steel was increased 50%, and rods 1 in. in diameter were spaced at 1 ft. These rods are embedded in a 6-in. pavement of concrete depressed 3 ft. at the middle of the stream so as to confine the current and protect the banks, and also to limit the size of the abutments, which would necessarily be much heavier if the springing were at a greater elevation above the pavement. The abutment ties are readily anchored in the abutments by extending them to near the back face and then bending them sidewise to overlap the adjacent ties.

With the arch-rods spaced at the same distance, the diameter of rod required is,

$$A = \sqrt{1 \times \frac{80 \times 20}{250 \times 12}} = 3/4 \text{ ins.}$$

Since the welds in the arch-rods may be alternated so that the bond of the concrete will protect the weakness of any one weld, the steel in the arch was not increased, but was composed of ¾ in. rods spaced at 12 ins. The arch-rods were bent to cross the arch ring at the middle and third points of the half arch, and were wired to the abutment ties at their ends to hold them to place during the concreting of the arch ring. All other details of the arch are the usual problems of wing and retaining walls, the spandrel walls being produced back of the curved wings to form guard-rails, and the wings reduced in height accordingly.

The footings for the abutments were placed first, then the concrete pavement with the abutment ties embedded and projecting at their ends over the footings. This pavement was laid one-half at a time, the stream being diverted across one-half of the pavement while the other half was being placed, by an earth cofferdam with the rods continued straight through the dam. Contrary to general belief, the rods do not cause leakage through such a dam.

The centers were then erected on the pavement, as shown in Fig. 9, consisting of 4 × 4 in. uprights spaced at 3 ft with roadway and at 8 ft. with span. The uprights were braced by sway bracing to prevent buckling; and centers were struck by removing the bracing and permitting the uprights to buckle.

Fig. 10 shows the bridge complete, except for slight additional filling required to bring the approaches to grade. This bridge has now been in service for nearly two years and has given the best of satisfaction.

The contract price for the bridge complete was $2,695. The amount of concrete required was approximately 265 cu. yds.; 4,500 lbs. of steel were used in the abutment ties, and 4,800 lbs. in the arch and spandrels. The steel consisted of smooth steel rods costing approximately 2 cts. per lb. in place. The steel required for the arch complete, cost less than 5% of the contract price of the structure; and if the steel in the abutment ties be not included (since it cannot properly be considered a part of the arch reinforcement) then the steel reinforcement for the arch and spandrels cost less than 3% of the total contract.

From the standpoint of efficiency, both of steel and of concrete, this type of reinforced arch shows some extremely favorable comparisons. The Wayne St. Bridge across the Wabash River at Peru, Ind. (Engineering News March

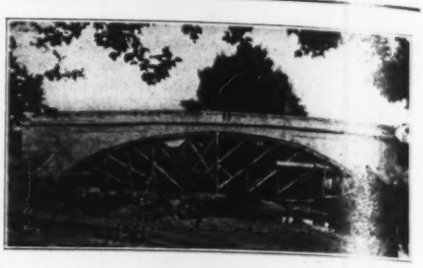


Fig. 9. Clifty Creek Arch with Centering in Place.

29, 1906), was designed according to this method, with abutment ties in the two end spans only, since all other thrusts are balanced; the steel reinforcement in arch and spandrels cost less than 3% of the contract price of the bridge.

Contrasting with these examples are the Charley Creek Bridge at Wabash, Ind., for which the arch reinforcement cost nearly 25% of the contract price; the Pollaskey Bridge near Fresno, Cal., for which the reinforcement cost upwards of 15% of the contract price; and the Meridian St. Bridge at Indianapolis, Ind., the reinforcement costing nearly 20% of the contract.

Since it might be supposed that a large percentage of reinforcement would accomplish a saving in concrete, the following dimensions and approximate contents of the above bridges are given, showing rather the reverse: Clifty Creek Bridge, span 80 ft., rise 12 ft., roadway 16 ft., concrete 265 cu. yds., contract $2,695. Wayne St. Bridge, six spans of 75 to 100 ft. each, rise 15 ft., roadway 30 ft., concrete 5,200 yds., contract $36,900. Charley Creek Bridge, two spans of 75 ft. each, rise 18 ft., roadway 30 ft., concrete 1,200 yds., contract $7,000. Pollaskey Bridge, ten spans of 75 ft. each, rise 11 ft., roadway 18 ft., concrete 3,000 yds., contract $18,000. Meridian St. Bridge, three spans of 74 ft. each, rise 9 ft., roadway 60 ft., concrete 5,000 yds., contract $51,000. To these may be added also the East Washington St. Bridge at Indianapolis, built according to the system described in this article, of span 65 ft., rise 10 ft., roadway 57 ft., concrete 1,100 yds., contract $10,885, for which the steel reinforcement of smooth rods and including the steel for abutment ties, cost less than 5% of the contract price. The contract prices for some of the bridges mentioned above are erratic; for example, that of the Pollaskey Bridge is very high, while that of the Charley Creek Bridge is low. But they are sufficient to illustrate the fact that there is great variation in the amount of steel required for reinforcement for highway bridges subjected to similar loadings, and that a system in which the steel costs less than 5% at the same time that it accomplishes the results, is an economical system.

The Clifty Creek Bridge and the East Washington St. Bridge mentioned above were built by the National Bridge Co., of Indianapolis.

FIG. 10. CLIFTY CREEK BRIDGE COMPLETED.

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The lock canal plan for Panama was approved by the Senate on June 21 by a vote of 36 to 31, and the Kittredge sea-level canal bill was then so amended as to provide for a lock canal in the following terms:

That a lock canal be constructed across the Isthmus of Panama, connecting the waters from the Atlantic to the Pacific oceans, of the general type proposed by the minority of the board of consulting engineers, created by order of the President, dated Jan. 24, 1905, in pursuance of an act entitled "An act to provide for the construction of a canal connecting the waters of the Atlantic and the Pacific oceans," approved Jan. 28, 1902.

With the House already strongly committed to the lock canal plan, as reported last week, and the Administration united in its favor, there can be no further question, apparently, as to the type of canal to be built at Panama. Chief Engineer Stevens and his staff will, therefore, proceed at once, as actively as the rainy season now in progress will permit, with the work on the lock canal plan. That means, doubtless, that excavation will promptly go forward on the site for the Gatun locks, for the construction of these locks, as our readers will recall, is likely to take nearly as long as the Culebra cut excavation. A force of engineers, headed by Mr. Joseph Ripley, M. Am. Soc. C. E., who has long been in charge of the Poe lock at the Sault, will at once be organized and proceed to prepare the detail plans for the great locks with their gates and operating machinery. It was really this work, more than anything else, that made a prompt decision on the canal-type imperative. At Culebra, digging could have proceeded for some time without much regard to whether the final summit level were to be at + 85 ft. or at ± 0 ft.; but every month's delay in beginning work on the locks may mean a month of delay in the completion of the canal.

The engineering profession as a whole, we believe, will welcome with satisfaction the final settlement of this question; and even those who have advocated the sea-level plan will in time, doubtless, come to recognize the sound wisdom of the decision that has just been made.

As we have said before in this place, there is no reasonable doubt that the engineering profession is fully equal to the task of building either a sea-level or a summit-level canal at Panama which would be thoroughly satisfactory for navigation purposes; but there are no demonstrated advantages of the sea-level project to justify the enormous additional outlay in time and money which it would inevitably involve. This fact was made so clear in the Senate debate that a number of Senators who had formed an opinion favorable to the sea-level plan, without having carefully studied the subject, were induced to change their position and record their votes in favor of the lock canal plan.

The daily press has taken little part in the struggle over the canal type—a commendable position in view of the lack of technical knowledge of the situation by those who have not given it careful study. It is worth while, however, for engineers to understand how the decision impresses the lay public, and we therefore reprint the following extract from the leading editorial in the New York "Journal of Commerce" of June 23:

Whatever lingering doubt there may be in many minds as to the ideal or the ultimate type of a channel connecting the oceans across that narrow but obstinate neck of land which joins the two Western continents, this prompt decision under present circumstances must be deemed fortunate.

The arguments in this case have been so evenly balanced, and the highest expert opinion so divided, that the lay mind may be excused for vacillating in regard to the merits of the two plans. There had been pretty full hearings before the committee and a careful study of the problem by a number of able men in the Senate, who made effective speeches, and the drift of conviction seemed to be toward the lock plan. If minds were changed by argument, that is a favorable indication, for consistent adhesion to a hastily formed and imperfectly supported opinion is far from being a merit. Another influence was probably the attitude of President Roosevelt, Secretary Taft, Chairman Shonts and all but one member of the Canal Commission, and of the chief engineer in charge of the work. Not only was their attitude an important factor in itself, but it was worthy of special respect, for these men have been for some time in a position of direct responsibility which impelled them to give close attention to all the considerations bearing upon the problem and to base their conclusions upon solid ground. The chance that they were right was much greater than that the judgment of the average Senator or Congressman was sounder than theirs. As to engineering opinion in general, including that of the majority members of the consulting board, it could not be said that there was a preponderance on the side of the sea-level canal.

It is fair to believe that the majority were persuaded by such arguments as were submitted in speeches like those of Senator Knox and Senator Cullom that the lock plan was really preferable under all the circumstances and conditions of the case. Senator Knox's speech was peculiarly calculated to carry conviction, not only because it was an able and forcible presentation of the case, but because, starting with his mind in favor of the sea-level canal, he had himself become convinced by a careful study of the question that the other plan had most in its favor.

That a lock canal is entirely feasible and can be successfully constructed and operated is doubted by few; that it can be completed in a much shorter time and at much less cost than a sea-level canal is not questioned by any. Some believe that on its own merits, apart from the consideration of time and cost, and as a permanent means of communication between the oceans, it is preferable and that experience will justify its adoption. If it should appear otherwise in the course of time, it will still be possible to cut the waterway down to the sea level at much less cost than if the work for a lock canal had not been done.

Accidents to large steel buildings are called to mind by the reference, in another column, to a number of large exhibition halls or convention buildings, and it may be of interest briefly to refer to the accidents which have occurred to some of these buildings. The two Chicago "Coliseums" of 1895 and 1899 both collapsed during the erection of the steel framework. This was due probably to an insufficient amount of the longitudinal bracing being in place or securely riveted, at a time when severe strains were imposed by high winds, by erecting machinery or by irregular loads of roof material stacked upon the erected framing. In each case the structure was rebuilt upon the original plans, but the old one (at 63d St.) was destroyed by fire the year after its completion and was not again rebuilt. The Kansas City convention hall was completely destroyed by fire a little more than a year after its opening, but was rebuilt in the short space of 47 days in order that the owners might fulfill their contract to accommodate a political convention. The Manufactures' Building at the Chicago Exhibition of 1893 was also destroyed by fire, but this occurred after the close of the exhibition, and was regarded with equanimity by the owners, for the reason that they were under obligations to remove it, and to take it down piecemeal would have been an expensive matter. In each of these three fires the steelwork fell and was a total and practically useless wreck, while the heaps of twisted and distorted steel made the clearing of the wreck a difficult matter. It may at first seem strange that these structures should collapse so suddenly in case of fire. The Chicago Coliseum is said to have been a wreck within 15 minutes after the fire started. As a matter of fact, however, while the buildings usually have brick or stone walls, they generally contain a large amount of inflammable matter in the shape of flooring, wainscoting, platforms, seats, interior finish and decorative work. During meetings or exhibitions this is, of course, largely increased by temporary structures and booths, signs, packing boxes, straw, paper, etc. When this is once well started it will make a quick, hot fire, which is sure to attack some part of the steelwork. This then buckles or gives way under load, and the collapse of the structure will rapidly follow. In regard to failure of the Chicago Coliseum buildings during erection, a somewhat similar case occurred at the new Chicago plant of the Western Electric Co. in 1903, when the nearly completed steel framework of the machine shop fell to the ground. Other similar failures have occurred with high buildings of structural steel construction. Accidents of this kind are, however, very rare in view of the enormous amount of steel building construction that has been done in the past ten years. When they do occur they are usually due in part to the fact that such construction (in all kinds of buildings) is often put up largely with a view to speed in erection; insufficient permanent or temporary bracing and loosely bolted connections are then usually found to be among the causes of accident, the erectors having "taken chances" once too often.

The Salton Sea, described in our issues of Feb. 22 and May 10, is still receiving the full discharge of the Colorado River, in which June floods are now flowing. The level of the sea, which was being raised only at the rate of a fraction of an inch a day in January, when the river was low, was increased by 3 ft. 4 ins. during the week ending June 2, and during the following week it was rising at the rate of nearly a foot a day, with only about 15 ft. more to rise before reaching the new location of the Southern Pacific tracks. Apparently the Colorado discharge is in excess of 100,000 cu. ft. per second, and its new outlet to the Salton Sea is likely to be eroded so deeply by this heavy flood that the task of restoring the river to its old bed, which will have to be undertaken anew when the flood ceases, will be still more difficult and expensive. The whole affair is a most remarkable example of blunderers meddling with great natural forces, producing damage which they are impotent to remedy.

In some Eastern states it has been found advisable to have state supervision of dams and reservoirs, to protect the public against danger due to irresponsible and incompetent engineering work. The Salton flood would indicate that the West may find it necessary also to bring about state supervision of irrigation and other works, not alone to secure the equitable distribution of water, but to protect the public against danger and loss.

The railway conventions held at Atlantic City, N. J., last week and the week before are likely to remain notable in the minds of those who were fortunate enough to be there, chiefly for the remarkable attendance. With only slight fluctuations from day to day, the convention attendance averaged 4,000 persons. The number of exhibits of railway appliances was half a hundred greater than ever before; indeed, setting aside the great world's fairs, it is probable that so large an exhibition of railway motive

power fittings and appliances has never been gathered in one place except during the International Railway Congress at Washington last year. Not only was the number of exhibits greater than ever before, but the size of individual exhibits was larger. In many cases power was laid on and the various tools and appliances were shown, not only in working order, but actually in operation. The educational value of such a representation of the latest developments in car and locomotive appliances and in shop tools to the master mechanic and master car builder is evident. Looked at from another direction, it illustrates what may be accomplished by the supplyman in the way of legitimate aid to technical societies in developing their convention attendance and providing useful occupation between sessions for their members.

The feature that stands out prominently in the technical work of the two motive power associations at Atlantic City is the role played by the testing laboratory. To itemize: the reports on locomotive front end design, on brake shoe friction, on brake beam efficiency, on strength and durability of air brake hose, on internal friction of locomotives and on locomotive efficiency, were all based on tests carried out at well-equipped testing plants. It is worthy of note that all but one of these sets of tests, namely, the locomotive tests of the Pennsylvania R. R. at the St. Louis Exposition, were made at what may for all practical purposes be termed the associations' own testing laboratory. The laboratory at Purdue University now contains all the testing apparatus belonging to the two associations, and by their close relations with the University it may be said with substantial truth that practically the whole laboratory equipment of the institution is at the disposal of the associations by payment of the bare cost of operation. It may be mentioned also that the associations are constantly increasing the testing apparatus owned by them; at the present sessions, for example, the air-brake testing rack was authorized to be enlarged from 50 to 100 sets, and it was recommended that the brake shoe testing machine be so remodeled that endurance as well as friction tests could be made.

Turning to the results of the year's work with the associations' testing plant two items stand out prominently. They are the brake beam tests by the Master Car Builders and the locomotive front end tests of the Master Mechanics. At the present time the Master Car Builders' Association's requirements for brake beams is based on the car weights and capacities of 17 years ago. Then a 30-ton capacity car weighing 25,000 lbs. was the average freight car; to-day cars are in extensive use with capacities of 55 tons and dead weights reaching 25 tons. A brake beam suitable to the loads of 17 years ago cannot obviously be suited to loads twice as great, but the surprising fact was brought out in the tests that many commercial beams now being applied to the heavier modern cars did not meet the requirements established by the association in 1889. No better proof than this is needed of the usefulness of the investigations conducted by the association during the past year. As the result of these investigations the committee recommended to the association an entirely new set of specifications for brake beams and their application, and if these recommendations are properly carried out they should result in a vast improvement in the present unsatisfactory conditions in freight car braking.

The tests on locomotive front ends presented to the Master Mechanics' Association are the third series of tests that have been made under its direction to settle the important question of the best arrangement and design of locomotive smoke-box and draft appliances. The first series of tests was made in 1896, and the second in 1902-3. Each series has developed important results. The tests made in 1896 at the Chicago & Northwestern Ry. testing plant give directions for the proper proportioning of exhaust nozzles; the tests made at Purdue University in 1902-3 developed a general equation for proportioning stacks, and the tests made at Purdue during the last year cleared up all disputed points and gave equations for the design of complete smoke-box and draft appliances. In conclusion, this series of investigations by its magnitude and thoroughness, and by the practical results secured, places the testing laboratory on the plane of an indispensable factor in the development of the mechanical department of our railways, and this series of tests is but one of a half dozen series reported at the railway conventions of 1906.

CONCERNING THE VARYING VALUE OF THE DOLLAR.

Forty years ago, at the close of the Civil War, the United States was doing business with a depreciated paper currency, worth only half and at times even less than half of its face value in gold. While the situation was clearly understood by the financial experts of that day, it was not understood by people in general. Buying and selling, lending and borrowing, went on as if the dollar were an unvarying standard of value, and with no foresight of the impending change. The debtor who borrowed a thousand dollars for a term of years had seldom any idea that he would actually at the end of the term have to pay the creditor in dollars twice as valuable as those he had originally received. The hardships growing out of this change in the value of the dollar were enormous and widespread. They were in large degree responsible for the greenback craze of a decade later and for other financial vagaries which have afflicted us since. No one who clearly understands the situation resulting from the depreciation of paper money in the '60's and '70's can doubt that a currency of fluctuating value is one of the worst evils that can befall a people.

While the financial issue on which public attention was concentrated a few years ago did much to educate the public in the elements of sound finance, there is one fact now pressing on public attention which was rather obscured in the discussion. We refer to the fact that the dollar, even when based on the gold standard, is not by any means an unvarying standard of value.

We shall not here discuss the so-called quantitative theory of money, further than to say that the enormous production of gold during the past twenty years does appear to have been one important factor in the depreciation of the dollar which has recently occurred. Theorists have often speculated as to what the result would be if a deposit of gold should be somewhere uncovered from which the metal could be produced in unlimited quantities at a labor cost much below the present value of gold. It is evident enough, of course, that such an event would absolutely reduce the purchasing power of gold everywhere; and to a certain extent, it is claimed, the great production of gold in South Africa, Alaska and Colorado has tended to produce a similar result.

That a great change in the value of the dollar has occurred is apparent to even the dullest observer. The value of money is measured, of course, by what it will procure. Dollars are worthless except as a universally accepted medium of exchange. If a dollar to-day will buy no more in food, clothing, shelter, personal service or other commodities than half a dollar would procure half a century ago, then it is a fair conclusion that the value of the dollar has diminished by one-half. One has but to refer to records of the first half century, showing prices paid and cost of living in those days to be absolutely convinced that a great change has taken place in the value of the dollar in that period. We do not, however, need to go so far back by any means. In the current number of "Moody's Magazine," a writer compares the average commodity prices in 1897, at the end of a long period of financial depression, and those of the current year. According to these figures, as compiled by R. G. Dun & Co., there was an average increase in price in that period, nine years, of 47%. That means that it now takes $1.47 to buy what $1 would have bought nine years ago.

It is curious to note that this rapid change in the value of the dollar (and the [illegible] dollar, too) has exactly reversed the situation between debtors and creditors that existed in the early '70's. Then the creditor got back from the debtor—if the debtor remained solvent—much more than he originally lent. To-day, a man loaned money nine years ago at 5%, and were now to be paid back the principal with simple interest, he could not purchase as much with the whole as he could with the principal alone when he lent the money. In other words, the shrinkage in the value of the dollar is more than offset all the interest it has earned.

This decline in the value of the dollar must not be confused with another decline which has been going on at the same time, the decline in the rates of interest; yet to a certain extent the two react upon and influence each other. Twenty-five years ago a hundred dollars loaned would earn six dollars interest every year. Now a hundred dollars loaned will earn only three dollars and a half a year; and that three dollars and a half will only purchase as much as a dollar and seventy-five cents did at that time. The "bloated bondholder," who has so long been held up to scorn, therefore, is now actually receiving from the same principal an income less than a third as great as that which he enjoyed twenty-five years ago.

It has seemed to us that these facts are worth bringing to the attention of engineers. While they may, in a way, be well known, we are all too prone to forget them. We unconsciously think of the dollar as a standard of value; but if it is a standard it is one whose dimensions are varying like a piece of india-rubber. We think of wages and salaries as if the dollars could be compared with the dollars paid in wages and salaries twenty years or even ten years ago. It needs but the least thought to see that this is not at all the case. The man, be he president, chief engineer, college professor, surveyor, blacksmith or ordinary laborer, who is paid the same number of dollars per day or per year that he was nine years ago, has actually suffered a reduction in his salary or wages of nearly one-third. He can actually buy only two-thirds of the necessaries or comforts of life to-day that he could nine years ago. On the other hand, these changes of values are creating riches on every hand. Those whose property consists in actual things—real estate, railways, ships, mines, stores and what not, have often seen a jump in the value of their holdings which was due only partially to their shrewd business judgment and largely to the fact that the dollar has depreciated in value and thus made their property worth more dollars.

Will there be a return to the lower prices of a former day? So far as the value of the dollar is influenced by the rate of gold production, there is no prospect of any reduction in the output of the world's mines. Rather, with the constant exploration of new countries and the rapid development of chemical and mechanical processes for treating low grade ores, a steady increase in the world's gold production seems probable, at least for a long period to come. There are, moreover, causes tending toward higher prices for various commodities, such as the growing scarcity of lumber and various metals, or the inability of the sources of supply to keep pace with the expansion of demand. It will be understood, of course, that an increase in price of any important commodity, to whatever market conditions it may be due, operates to decrease the value of the dollar. Inevitably, too, there must be a further readjustment of wages and salaries in many departments to correspond with changed conditions above set forth. All these things tend to make permanent the decreased purchasing power of the dollar and to bring about still further decrease.

A question of very practical import relates to the policy which should be pursued in making investments not only by private persons but by the trustees of institutions, such as schools, colleges, savings banks, etc. There can be no doubt that much of the popular favor with which

bonds are regarded by the public is born of past experience. But if bonds—or promises to pay in dollars—are subject to serious loss through depreciation of the dollar, may there not be a change in the public temper? May not the shares of stock in companies having no bonded indebtedness or fixed charges replace in popular favor the issue of bonds. The question is one worth the serious attention of financiers.

LETTERS TO THE EDITOR.

Effect of Mica in Sand for Concrete.

Sir: I would be very glad if you could give me any information or refer me to any article giving report of tests showing effect of mica in sand used for concrete or mortar. I am making some tests and would like to know if any one has reported any tests on this subject, so that I may compare results.

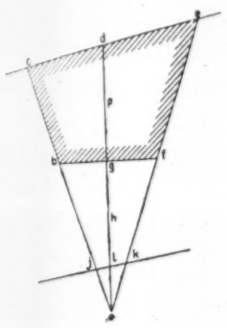
Yours very truly,

Bristol, Va.-Tenn., June 15, 1906. W. N. Willis.

A Method of Computing Excavation and Embankment.

Sir: The method of figuring the amount of excavation and embankment, for a preliminary estimate of a railroad line, by using slips of profile paper with the proper quantity written opposite each foot, directly applied to the profile is well known. The preparation of such slips, for the various slopes of ground, rate of side slopes and different widths of roadbed involves a great deal of labor. The writer does not know whether the following method for obtaining the data for these slips has ever been set forth before, but gives it for what it is worth, in the hope that, whether original or not, it may be found useful by some engineers engaged in this work.

To find the area of any cross-section; where slope of ground, width of roadbed, rate of side slopes and center height is given.



In the accompanying figure let b c e f = A, the area required.

Let the side slopes f e and b c = s (rate).

Let the rate of ground slope c e = s′.

Let p be any center height of cut.

Produce the side slopes until they meet at o. The line of center height produced d g o will also pass through o.

Let o g = h.

Let area o b f = Q.

Draw j l k parallel to c e making o l = 1 ft.

Let area o j k = a.

The triangles o j k and o c e are similar. Therefore

$$\frac{A+Q}{a} = \frac{\overline{h+p}^2}{\overline{o\,l}^2} = \frac{\overline{h+p}^2}{1} \text{ or } A = a(h+p)^2 - Q \ldots (1)$$

Say a table is required where the ground slope is 30 in 100 ft., the side slopes ½ to 1, and the roadbed 20 ft. wide. Then in equation (1) a, h and Q are constants and we find from the triangle o b f, h = 20 ft. and Q = 200 sq. ft.

To find the area of o j k = a: Let o l = axis of Y with the origin at O. Then the equation of line o k is

$$y = s\,x \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The equation of line j l k referred to the same axis and origin is $y' = s'x' + c'$, but in this case the constant $c' = 1$ since o l was laid off equal to 1 ft., or

$$y' = s'x' + 1 \ldots\ldots\ldots\ldots (3)$$

Substituting values of s and s′ in (2) and (3) we have $y = 2x$ and $y' = \frac{30}{100}x' + 1$. At k, evidently, x = x′ and y = y′, so combining (2) and (3) we have $2x = \frac{3}{10}x + 1$ or x = .5882 ft. This x is evidently the altitude of the triangle o l k. Similarly for the point j, x = — .4348 ft., the (—) sign denoting its relation to the y axis, or the area of

$$o\,j\,k = \frac{(.5882 + .4348) \times 1}{2} = a = .5115 \text{ sq. ft.}$$

Now substituting in (1), the values found, we have $A = .5115\,(20 + p)^2 - 200$ sq. ft.

Let R = cu. yds. per 100 ft.

then $R = \frac{51.15}{27}(20 + p)^2 - \frac{20{,}000}{27}$ cu. yds. or

$$R = 1.8944\,(20 + p)^2 - 741 \text{ cu. yds.}$$

Now give p values increasing by 1 ft. each time, work out three values and so obtain the second difference by which all the remaining values may be written out and the table made up, or with a table of squares and the use of the slide rule or logarithms work each foot out independently. In using the second difference method it is well to work out independently the value at every 10 ft. to check the work. It is evident that when the line of ground slope cuts the plane of the roadbed the method fails, but these few values in this case up to 3 ft. center height may be easily found.

Yours truly, Ewing D. Sloan,
Asst. Engr., Tidewater Ry. Co.

Norfolk, Va., May 21, 1906.

INVESTIGATION OF THE THERMAL CONDUCTIVITY OF CONCRETE AND THE EFFECT OF HEAT UPON ITS STRENGTH AND ELASTIC PROPERTIES.*

By Ira H. Woolson,† E. M.

This paper is a record of some experiments made in continuation of work described in a previous paper‡ before the American Society for Testing Materials. That paper recorded a series of tests showing that both the strength and elasticity of concrete were rapidly diminished by the application of comparatively low heat for moderate periods of time. The results appeared quite contradictory to well established evidence regarding the behavior of that material in actual fires, and in fire tests of full size floor units. Endeavoring to elucidate this discrepancy, the writer made a single experiment on the conductivity of concrete which apparently explained the whole variation, and pointed to the fact that the virtue of concrete as a fire resisting material was due to its high non-conductivity. The other tests showed that if it were thoroughly heated to 1,500° F. its strength and elasticity were largely dissipated.

In the discussion which followed the reading of the paper it was suggested that further investigation of the conductivity of concrete should be made; also, that additional proof of the other results, using larger specimens and different concrete mixtures, was desirable. At the request of the "Joint Committee on Concrete" the writer has continued the experiments and this paper is a summary of the new data.

The actual work of preparing and testing the specimens, making the calculations, and plotting the curves was done by students in the Department of Mechanical Engineering, Columbia University, under the writer's direction. This year the investigation was made by Messrs. R. M. Olyphant, Jr., Geo. W. Riddle, and W. Putnam, students in the senior class, who did the work as a thesis problem. Their hearty cooperation and faithful attention to detail has alone made this paper possible.

The composition used was the same as last year, namely, a 1-2-4 mixture of Portland cement, sharp sand and ¾-in. clean broken stone, both trap rock and limestone being employed. In addition to these, other specimens were made in the same proportions, using clean ½-in. quartz gravel instead of the broken stone, and in another set, clean boiler cinders instead of stone. The cement was furnished by the committee and was said to be composed of a mixture of "Giant," "Lehigh" and "Vulcanite." It was a slow-setting cement taking 7 hrs.-and 40 min. for initial set, and 14 hrs.-10 min. for hard set. The average strength neat for 1 day was 246 lbs.; 7 days was 710 lbs. Mixed with three parts sand, the average for 7 days was 160 lbs., and for 28 days, 286 lbs. The concrete was mixed moderately wet and well tamped. The specimens were left in the molds 36 hours, then removed and submerged in water for 7 days, after which they were allowed to stand for 7 weeks in a cellar, being occasionally sprinkled.

It was suggested that instead of raising the temperature of the specimens very slowly, so that three or four hours were consumed in attaining a furnace temperature of 1,500 to 2,000° F., as was done last year, it would be instructive to bring the furnace temperature rapidly up to some fixed point, then holding it there for a definite period, the idea being that duplicate tests would be easier and the conditions approach more nearly those of an actual fire. This was done in all cases in the present series of experiments. 1,500° F. was adopted as a fair average, and the furnace raised to that temperature in 40 to 60 minutes and held there for the balance of the test. It was thought unwise to raise the temperature faster than this for fear of producing internal stresses that might rupture the specimens. In one instance the furnace did not work properly and the necessary heat was not secured in an hour. Three sizes of specimens were made: 4-in. cubes, 7-in. cubes and 6-in. × 6-in. × 14-in. prisms. The latter were used for measuring the elastic properties. The 4-in. cubes were made for comparing the strength of the new concrete with that of last year.

METHOD OF HEATING.—The specimens were heated in an oven-type of gas furnace. Those placed entirely inside the furnace were protected from the direct impact of the flames on the sides by shields of fire brick or clay tiles. This was done to insure as nearly as possible an even distribution of heat throughout the whole specimen.

Three and five hours were chosen for the times of heating the large specimens, and two and three hours for the 4-in. cubes. It was estimated that the internal temperature of the small specimens in that time would be about the same as that of the large specimens exposed the longer time. These time periods include that required for the furnace to reach 1,500° F. After heating, the specimens were removed from the furnace and allowed to cool slowly in air.

The temperatures were measured by a carefully standardized Le Chatelier pyrometer, readings being taken simultaneously every ten minutes upon the thermocouple placed between the specimens, which registered the temperature of the furnace, and upon those embedded in the specimens which gave the corresponding internal temperatures at the points taken.

The specimens used to measure the conductivity by heating upon one side only, were placed in the doorway of the furnace with the exposed face on a line with the furnace wall. These specimens completely filled the doorway with the exception of an inch at the top which was filled with magnesite board. Fig. 1 is a sketch showing arrangement of large specimens in the furnace and in doorway.

Specimens designed for strength tests were faced with plaster-of-paris before testing.

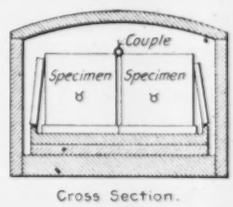
A few tests were made upon some specimens left over from last year's experiments, and two or three upon some samples of 1-2-3 trap rock mixtures two years old.

CLASSIFICATION OF SPECIMENS.—

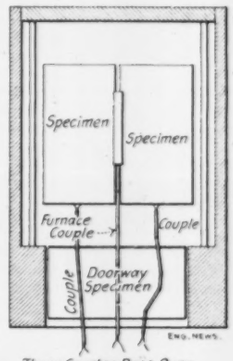
Lot A	1-2-4 mixture,	limestone,	age 2 months.
Lot B	1-2-4 mixture,	trap rock,	age 2 months.
Lot C	1-2-4 mixture,	cinder,	age 2 months.
Lot D	1-2-4 mixture,	gravel,	age 2 months.
Lot E	1-2-4 mixture,	limestone,	age 1 year.
Lot F	1-2-4 mixture,	trap rock,	age 1 year.
Lot H	1-2-4 mixture,	trap rock,	age 1 year.
Lot K	1-2-3 mixture,	trap rock,	age 2 years.

CONDUCTIVITY TESTS.

Most of the specimens used in this investigation were 7-in. cubes, this being as large as the furnace would take. A few were 6 × 6 × 14-in. prisms. When these specimens were made, thermocouples were cast in them at varying distances from the surface, the idea being that when these were tested in the furnace and temperature readings taken simultaneously upon the furnace atmosphere, and the couples embedded at varying depths in the concrete, the rate of transmission of heat through the concrete would be obtained. The work was carefully done, but results were confusing, and subsequent breaking of the specimens showed that the couples had been shifted by tamping the concrete when the specimens were made. In some cases they were an inch out of place and this completely nullified the work in regard to obtaining a rate of transmission from these specimens. Figs. 2 to 5 are the curves plotted from these tests for a period of five hours in each instance. One curve gives the temperature of the furnace, which was brought up to 1,500° F. in less than an hour and held there for the balance of the test, and the other curves show the corresponding temperatures on the interior of the specimens. Although the depths given to the couples are not entirely reliable for reasons already explained, nevertheless the curves are very interesting in showing the general lag in the rise of temperature in the specimen as compared with the furnace temperature.



Cross Section.



These Couples Pass Over Specimen in Doorway

Plan.

Fig. 1. Sketch of Arrangement of Large (6 × 6 × 14 Ins.) Specimens in Furnace.

The gravel specimens were the only ones which attained an interior temperature equal to the furnace temperature.

It is rather surprising to note that cinder came next to the gravel in the amount of interior heat recorded, for cinder concrete is well known to be an effective fire resistant. However, the cinder concrete specimen, tested in the furnace doorway, gave the lowest interior heat. It was found impossible with the small furnace employed to maintain an even distribution of heat, and undoubtedly some of the conflicting testimony of the tests was attributable to this cause.

The curves in Fig. 6 are from tests of specimens one

*A paper read before the American Society for Testing Materials at Atlantic City, N. J., June 21, 1906.

†Adjunct Professor of Mechanical Engineering, Columbia University, New York, N. Y.

‡"Investigation of the Effect of Heat Upon the Crushing Strength and Elastic Properties of Concrete." Ira H. Woolson, Eng. News, July 13, 1905, pp. 35-38.

and two years old, and although the furnace worked badly, thus giving a very slow temperature rise, there is no evidence that either specimen possessed heat-resisting qualities superior to those of the new concretes.

Attention is called to one peculiar characteristic in all the curves. This is the flattening out at or a little after the 212° point is past, showing that after steam begins to generate there is little if any rise in temperature until all the water in the concrete has been evaporated. The only curve in which this peculiarity is absent is K 1, Fig. 6, and this specimen was two years old; it had been kept in a very dry room for over a year, and probably contained very little moisture. This was the only specimen available at this age and unfortunately it was tested when the furnace worked badly so the results are not conclusive.

After ascertaining that the locations of the thermocouples in the above described specimens were not all exact, one special trap rock block was made having seven holes ¼-in. in diameter cored in as shown in Fig. 7. Th bottoms of the holes were arranged from 1 to 7 ins. from the face to be exposed to heat, and the holes were spaced 1½ ins. apart. Thermocouples were sealed in these holes and all connected through a switchboard to the same galvanometer. The position of these thermocouples was absolutely assured.

This specimen was placed in the furnace doorway and subjected to 1,500° F. on one face for five hours (see Fig. 8). Readings were taken every ten minutes on each thermocouple, and the results are shown in the plotted temperature curves in Fig. 9. These curves are very instructive although there is some interference. The first salient characteristic of them all is the lag at the 212° point. It will be noted that this is greater with increased thickness of the concrete; also that up to 3 ins. thickness the recorded temperature at which it occurs is above 212°, showing that the surface of the concrete heated very rapidly and went above the boiling point of water. It is undoubtedly these conditions which cause steam to generate rapidly enough to become explosive and burst off patches of concrete, which is a common occurrence in fire tests.

The curves for the points 1 in. and 2 ins. from the heated face are nearly the same. The indications are that they would have been run together had the test continued longer. The reason for this is not apparent, but it may have been due to an accidental arrangement of the stone in the concrete in such a way as to form a comparatively solid stone connection between the thermocouples, which by its better conducting qualities tended to equalize the temperature between the points.

Considering these two curves as one, there is a marked parallelism between it and the curves embedded 3, 4 and 5 ins. in the concrete, and the distance between them is very constant. The curves for 6 and 7-in. points are irregular, although No. 7 is nearly parallel with the others. No. 6 is particularly freakish. The reason for this is plain. Points Nos. 6 and 7 were 2¼ ins. from the end of the block (see Fig. 7) and as these ends were more or less heated from the furnace walls, the points in question received some heat from this source as well as the face. This threw their temperature curves out of harmony with the central points which were not so affected.

As a whole, they are regular and indicate that the heat-resisting quality is not a direct function of the thickness. It will be noted that the curves flatten out and that doubling the thickness more than doubled the time required to secure the same temperature rise.

These curves as well as those of Figs. 2 and 6 show conclusively that the very low conductivity of concrete indicated by the single test reported in our paper of last year was entirely correct. They prove that two or three inches of concrete properly mixed, tamped and set, will resist a fierce conflagration for hours without permitting a serious temperature rise upon the opposite side.

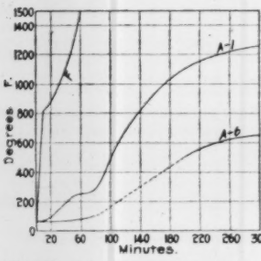


Fig. 2. Limestone Concrete, Two Months Old.

F. Furnace temperature.
A 1. Specimen in furnace, temperature measured 3 ins. from face.
A 6. Specimen in doorway, temperature measured 4½ ins. from face.

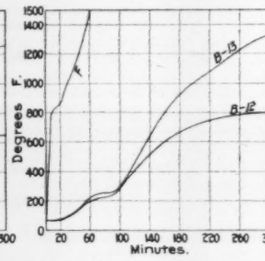


Fig. 3. Trap-Rock Concrete, Two Months Old.

F. Furnace temperature.
B 13. Specimen in furnace, temperature measured 1 in. from face.
B 12. Specimen in doorway, temperature measured 1½ ins. from face.

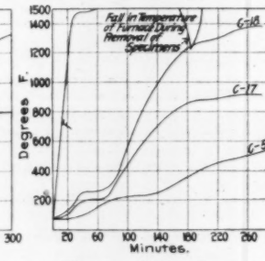


Fig. 4. Cinder Concrete, Two Months Old.

F. Furnace temperature.
C 18. Specimen in furnace, temperature measured 2 ins. from face.
C 17. Specimen in furnace, temperature measured 3 ins. from face.
C 5. Specimen in doorway, temperature measured 1½ ins. from face.

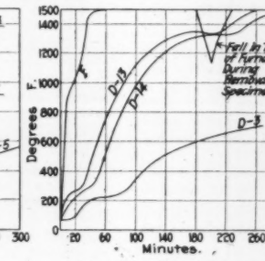


Fig. 5. Gravel Concrete, Two Months Old.

F. Furnace temperature.
D 13. Specimen in furnace, temperature measured 2 ins. from face.
D 14. Specimen in furnace, temperature measured 3 ins. from face.
D 3. Specimen in doorway, temperature measured 4½ ins. from face.

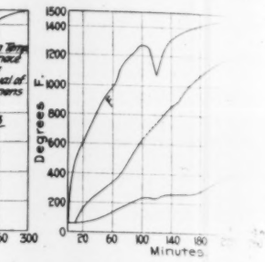


Fig. 6. Trap-Rock Concrete, Two Years Old.

F. Furnace temperature.
K 1. Specimen in furnace, temperature measured 3 ins. from face.
H 2. Specimen in doorway, temperature measured 4½ ins. from face.

FIGS. 2 TO 6. TEMPERATURE RISE AT DIFFERENT POINTS IN LIMESTONE, TRAP, CINDER AND GRAVEL CONCRETES (1:2:4 MIXTURES).

STRENGTH AND ELASTICITY TESTS.

Table I. gives the results of the tests upon the 4-in. cubes. In the one-year-old specimens (E and F) comparison is given with tests of duplicates made when they were one month old. The increase in strength is not large. Both the limestone and trap specimens made this year are much stronger than those made a year ago. The probable explanation is that the specimens were soaked in water this year for one week on removal from the molds, and last year were only sprinkled. It is doubtful if they then received sufficient water to produce a perfect set of the cement.

TABLE I—CRUSHING STRENGTH OF CONCRETE BEFORE AND AFTER HEATING.

Lot	Material.	Breaking Loads, Lbs. Per Sq. In. Not Heated.	Same lot in 1905 when 1 mo. old.	After heating 2 hrs. at 1,500° F.	After heating 3 hrs. at 1,500° F.	After heating 5 hrs. at 1,500° F.
		4 x 4 x 4 in. specimens.				
A	Limestone...	3,030	.....	980	1,445	
B	Trap Rock...	3,195	.....	1,300	1,410	
C	Cinder.....	1,360	.....	750	447	
D	Gravel.....	2,740				
E	Limestone 1 year old..	1,741	1,968	1,235	937	
		2,165	1,843	1,110	996	
			1,640	1,200		
				1,290		
F	Trap Rock 1 year old...	2,230	1,903	1,525	665	
		2,800	1,913	.....	695	
		2,240	1,892	.....	.....	4 hrs.
K	6 inch cube limestone 2 years old.	2,290	.....	.....	.....	2,005
		6 x 6 x 14 inch specimens.				
A	.....	2,740	.....	.....	1,345	870
B	.....	3,140	.....	.....	1,400	997
C	.....	1,400	.....	.....	547	504
D	.....	2,780	.....	.....	.....	.....

NOTE.—Where more than one breaking load is given it indicates that several specimens were tested.

It will be noted that specimens A and B three hours at 1,500° F. were stronger than two hours. This is contrary to all our perience and must be attributed to one of either the specimens heated two hours ha weak structurally, or the three-hour specim tected in some way in the furnace and did heat credited to them. In any event the anomalous and should be rejected.

No strength tests were made upon heated

Table II. gives the modulus of elasticity (E) calculated on increasing loads for specimens that had been heated for different times. Usually the modulus of unheated specimens decreases with increase of load, but it is not infrequent that they reverse the rule; the exception is in evidence in this series. The moduli of the heated specimens are very low in value, but in all cases the value increases with increase of load. This peculiarity was referred to last year. The reason is not clear, but perhaps it is because the specimen expanded on heating and when cooling, the outside contracts most rapidly, thus developing many fine cracks which do not entirely close when normal temperature is reached. The first loads applied to the specimen serve merely to close these cracks and the compression observed is not, therefore, the true deformation of the concrete for these pressures. The value of the moduli decreases very rapidly with increase of heat treatment. This fact was also brought out last year, but the present figures are merely confirmatory.

The limestone lost about 50% of its strength when heated 2 hours, and the trap about 55%. The difference was undoubtedly accidental, for we well know that trap is superior to limestone in resisting heat. They both lost 68% of their strength in 5 hours, when heated on all sides with no radiation permissible.

The strength of the 6 × 6 × 14-in. specimens heated on one face, see Table III., is in marked contrast to those heated on all sides, and shows that under that condition the concrete retains a large part of its strength even after five hours of exposure to a temperature of 1,500° F. As a matter of fact the specimens tested in this manner really received considerable heat upon four other faces, for it was impossible to protect them from the radiant heat from the furnace walls, so they were probably heated more thoroughly than they would have been had they formed a part of a concrete wall facing the fire.

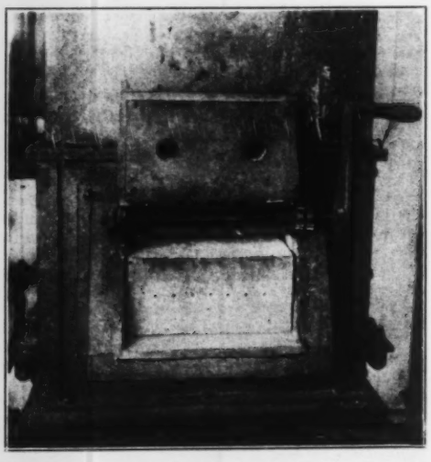


Fig. 8. View of Special Concrete Specimen in Place in Furnace Doorway (Packing Around Edges Removed).

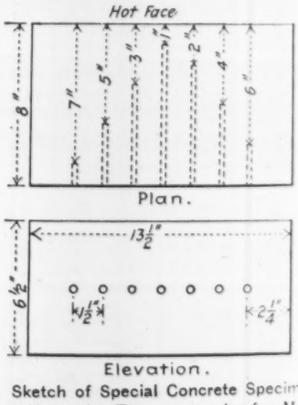


Fig. 7. Sketch of Special Concrete Specimen with Cored Holes for Thermo-couples for Measuring Heat Transmission.

TABLE II.—ELASTIC PROPERTIES OF 6 x 6 x 14 IN. PRISMS.

Lot Material.	E at 200 lbs. per sq. in. Unheated.	Heated 3 hrs.	Heated 5 hrs.	E at 400 lbs. per sq. in. Unheated.	Heated 2 hrs.	Heated 5 hrs.	E at 800 lbs. per sq. in. Unheated.	Heated 3 hrs.
A Limestone.....	6,000,000	200,000	.....	6,000,000	285,000	.....	5,647,000	425,000
B Trap Rock.....	3,430,000	150,000	129,000	4,355,000	222,000	188,000	4,355,000	348,000
C Gravel.....	1,090,000	49,500	57,100	960,000	.....	.....	915,000	
D Gravel.....	8,000,000	.....	.....	6,887,000	.....	.....	6,000,000	

Blank spaces indicate that specimens had failed by fire or crushing.

TABLE III.—ELASTIC PROPERTIES OF 6 x 6 x 14 IN. PRISMS HEATED ON ONE FACE ONLY.

Lot Material.	Modulus of Rupture (E) after heating 5 hours — At 200 lbs. per sq. in.	At 400 lbs. per sq. in.	At 800 lbs. per sq. in.	Breaking Load. Lbs. per sq. in.
A Limestone.....	293,400	521,700	730,700	1,840
B Trap Rock.....	200,000	268,000	379,000	1,705

Fig. 10 gives a specimen sheet of the elastic curves obtained from the 6 x 6 x 14-in. specimens both before and after heat treatment. Three curves are plotted for each test, (1) total deflection, (2) set, (3) true elastic; the last being obtained by subtracting the second from the first. (Negative sets were in evidence in these curves, though not so pronounced as in some previous tests.) The prominent feature of the curves is the great increase in deformation with loss of strength due to heat treatment. The elastic curves of the specimens which were one and

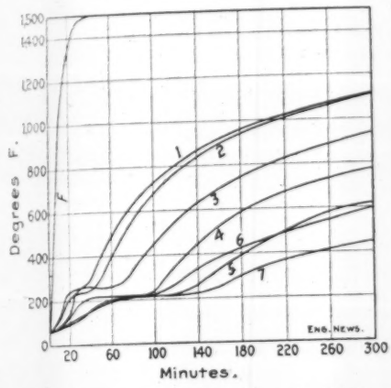


Fig. 9. Diagram of Temperatures in Interior of Special Concrete Specimen in Five-Hour Heating Test.

F = Furnace temperature.
Number on curve represents inches distance from heated face.

two years old, respectively, do not, apparently, show any material increase in their fire-resisting qualities, although these specimens were heated only 4 hrs. Fig. 11 gives the true elastic curves, grouped according to treatment, without regard to the total deflections from which they were derived.

The marked difference in the strength of the specimens made last year and this year, in which the same stone was used, and presumably an equal quality of cement, show that the treatment during the period of setting has much to do with the quality of the concrete. It would be interesting to ascertain if the fire-resisting properties are influenced by the amount of moisture absorbed during setting.

It is fully recognized that definite conclusions cannot be drawn from tests so incomplete as these; but they serve to point the way and it is hoped that more elaborate experiments may be made which will furnish the additional evidence necessary.

ANNUAL MEETING OF THE AMERICAN SOCIETY FOR TESTING MATERIALS AT ATLANTIC CITY, N. J.

The American Society for Testing Materials met in convention at Atlantic City, N. J., on June 21, 22 and 23. A large attendance had been promised and the promise was to a large degree fulfilled, there being some 200 members present. A programme of papers and committee reports had been prepared which was notable for its length; indeed, the amount of work laid down was too great for the best results in discussion, and as a matter of fact it was got through with only by energetic work and the judicious skipping of unimportant matters. In several instances also papers were presented only by title.

The convention opened with a brief session for transacting routine business and the election of officers. A canvass of the ballots showed that the present list of officers had been unanimously re-elected. The President's address, or, rather, the address of the Vice-President, Mr. R. W. Lesley, the President being unavoidably absent from the convention, was delivered at a later session; it was a historical review of the organization and growth of the society and its work, and covered matters familiar to most of our readers.

ROAD MATERIALS.

The report of Committee H on Standard Tests of Road Materials was not presented, so that the consideration of this class of materials was brief, consisting of a paper by Mr. A. S. Cushman on "The Development of the Test for the Cementing Value of Road Materials," and a paper by Mr. P. L. Wormeley, Jr., on "Hardness and Abrasion Tests of Road Materials." Mr. Cushman in his paper described briefly the original test worked out for the Massachusetts Highway Commission, in which stone dust produced by wear was molded with water into cylindrical briquettes that were tested by hammer blow or impact. The first development of this test was to substitute ground stone for road dust. This was followed by a change in gaging practice, the briquette, instead of being molded immediately after gaging, was not molded until the paste had been worked and kneaded a considerable time. A further development in this same direction was the production of the paste by wet grinding, or rather the use for the briquette of the slurry produced by grinding the stone samples with water in a ball mill. Test results were given, showing the strength increase due to long working and wet grinding and due to the mixing of stone of high cementitious value with stone of low cementitious value. The paper by Mr. Wormeley was a description of the apparatus used in testing road material stone for hardness and resistance to abrasion. Both papers were passed without discussion.

CEMENTS AND CONCRETE.

Next to the subjects of metals and specifications for metals, cement and concrete received the most attention. The programme may be said to have been begun at the first session by the report of Committee P on Fireproofing Materials, by Prof. Ira H. Woolson. In brief, the committee had confined its work during the year past to a collation of all available reliable data on tests of fireproof floors of reinforced concrete. Results were given in tabular form of 80 tests of such floors, 63 of these tests being selected from those made in years past by the Department of Buildings of New York City, and 17 being selected from tests made by the British Fire Prevention Committee. As the bulk of these tests have been recorded in Engineering News from time to time as they were made, they need not be further described at this time. In closing its report the committee presented a suggested design for a house for testing fire-resisting floors. The report received no discussion.

The paper on Concrete Column Tests at Watertown Arsenal, by Mr. Jas. E. Howard, was the second to be presented in the work of the section on cements and concretes. It described the results of tests of about 100 full size columns of plain and reinforced concrete, and will be published in abstract in a future issue of Engineering News. The paper was discussed briefly by Prof. A. N. Talbot and by Mr. R. A. Cummings. Mr. Cummings' discussion consisted of a statement of the results of a series of German tests on longitudinally reinforced and on hooped columns, the conclusion from which was that hooped reinforcing was materially the superior. Mr. Cummings also criticised the practice in the Watertown tests of having the longitudinal reinforcing bars the full length of the columns, so that the platens of the testing machine bore directly on them.

The succeeding paper by Prof. Ira H. Woolson on the Conductivity of Concrete and the Effect of Heat on Its Strength and Elastic Properties is reprinted on preceding pages of this issue. The discussion of this paper turned into a sort of experience meeting on the comparative value of gravel and broken stone for concrete aggregate; a number of members were disposed to question the value of gravel, and each cited some experience in support of his doubts. The notion that gravel did not make a good concrete aggregate was vigorously disputed by Mr. Richard L. Humphrey. Mr. Humphrey stated that the tests made by him for the Government on gravels from all parts of the country showed beyond doubt that it was quite as good as broken stone, in fact they showed that if there was any difference at all it was in favor of the gravel aggregate. Turning to the heat resistance of concrete, as shown by Prof. Woolson's tests, Prof. A. N. Talbot remarked on the severe character of the tests; he considered that the testing of small cubes by heating them to 1500° on all sides was such a trial as concrete was seldom subjected to in practice. His idea would be that a test should be developed in which a slab of concrete could be heated on one side only. Bearing on this suggestion, Prof. Woolson referred to accidental results obtained in testing a fireproof floor construction. In this test it was necessary to divide the test house by a partition. This partition was designed to be built of trap rock concrete, but by accident the workmen, after building up several feet of partition with trap rock, ran out of trap and substituted limestone and some 3 ft. in height of the wall was built with this material before the change was discovered and the trap rock mixture returned to. An examination of the partition after the floor test showed the trap rock portions of the wall

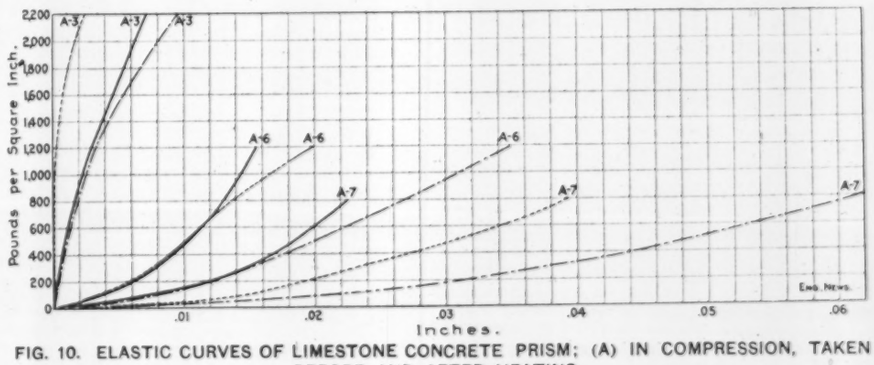


FIG. 10. ELASTIC CURVES OF LIMESTONE CONCRETE PRISM; (A) IN COMPRESSION, TAKEN BEFORE AND AFTER HEATING.

Dot-and-dash-line, total compression.
Dotted line, permanent set.
Full line, elastic compression.

Curves A 3, unheated specimen.
A 7, specimen heated in furnace 3 hrs.
A 6, specimen heated in doorway 5 hrs.

almost untouched by the fire, while the limestone belt was so pitted and disintegrated that it formed a broad groove in the face of the wall.

In a paper entitled a Note on Compression Tests of Cement, Mr. W. P. Taylor gave the results of some 3,000 tests on short cylinders of cement about 1 in. high and 1 sq. in. end area. This paper will be published in a future issue. It received somewhat extensive discussion, based on the shape and height of the test pieces. It was pointed out that the height of cylinder was less than half the height that theory required for securing true compression breaks. In answer, Mr. Taylor acknowledged the justice of this criticism but stated that comparative tests showed an almost constant and uniform ratio of difference, and having this as a guide for correcting results, he considered that the vastly greater ease of molding the smaller tests piece made its adoption justified.

Two other papers on cement testing were read, one by Mr. W. N. Forrest on A New Device for Mechanical Sifting of Concrete Aggregates. The device described consisted simply of a series of sieves set one above the other and ranging from coarse to fine and given a combined eccentric rotating and vertical jarring motion by a mechanism operated by a hand crank or power shaft. There was no discussion. Mr. R. K. Meade, in a paper on The Determination of the Specific Gravity of Cements, described a new apparatus for specific gravity tests and explained its operation in detail.

Turning from cements to reinforced concrete, Committee I on Reinforced Concrete reported progress of work. So far the work had been simply to collate the results of tests that had been made at various college laboratories under the committee's direction to plan for future work. In presenting its report the committee referred to the college work as being disappointing and leading to results of less value than had been expected. This characterization of the college work drew a strong protest from Prof. A. N. Talbot, who considered that on the whole very valuable work had been done by the college laboratories and that the committee had exhibited rather small appreciation of a very great amount of hard and faithful work. The discussion pro and con was somewhat prolonged and finally resulted in the agreement of the committee to modify its report in respect to the remarks objected to.

Papers on Tests of Reinforced Concrete Beams were read by Prof. Gaetano Lanza and by Prof. S. J. Tilden. Both papers described in detail a series of beam tests conducted by the authors.

PRESERVATIVE COATINGS.

Committee E, on Preservative Coatings for Iron and Steel, presented a progress report stating in brief that arrangements had been made by which 19 different metal paints were being applied to the Pennsylvania R. R. at Havre de Grace, Md. A 30-ft. length of the bridge was painted with each sample under the inspection of the committee's engineer. This report was followed by a paper by Mr. A. B. Harrison describing a new preservative coating for iron and steel that had been developed by the author and had shown excellent results in tests made by him. It developed in the discussion that this coating was one of those being applied to the Havre de Grace bridge. The discussion turned almost entirely on the best method of cleaning metal surfaces for painting, in which the method of sand blasting was unanimously condemned. In a brief paper on Practical Testing and Valuation of Japan, Mr. Robert Job described the test worked out by him for the purpose and now used by the Philadelphia & Reading R. R.

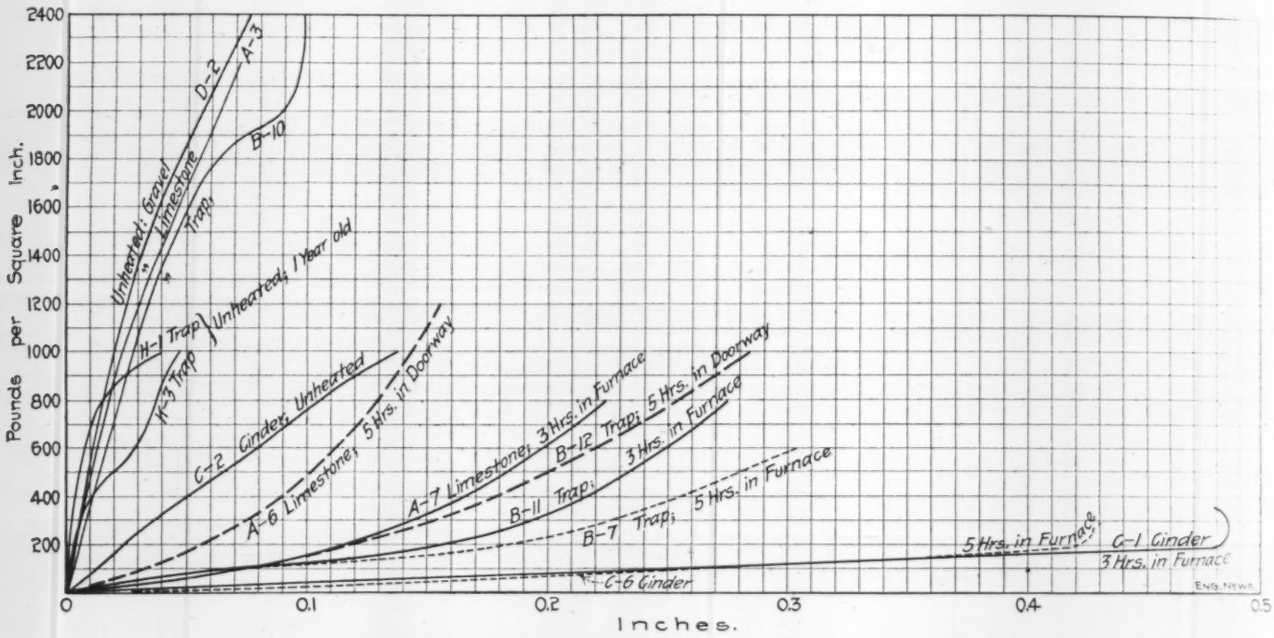


FIG. 11. CURVES OF ELASTIC COMPRESSION OF CONCRETE PRISMS, BEFORE AND AFTER EXPOSURE TO HEAT.

WATERPROOFING AND TESTING MACHINES.

The consideration of waterproofing was confined to the presentation of a progress report by Committee S on Waterproofing Materials and two papers, one Some Mistakes in Waterproofing, by E. W. DeKnight, and one on Methods of Testing Cements for Waterproofing Properties, by Mr. W. P. Taylor. In a succeeding issue extracts will be given of these papers. There were two complete papers on testing machines, one describing the Purdue University Impact Machine, by W. K. Hatt and W. P. Turner, and one on New Features of Two Large Testing Machines, by W. C. DuComb, Jr.

IRON AND STEEL.

The section dealing with Iron and Steel held three sessions, simultaneous with the sessions of the Cement and Concrete, Paints, and Asphalt sections. The section displayed much activity, and its discussions were of much interest; but no progress was recorded in the most important line of work, the making of specifications. Indeed, in the matter of Specifications for Steel Rails, the results are rather a retrogression. In the main this is due to the fact that the most important committee of the section, Committee A (on Standard Specifications for Iron and Steel) has finally proved to be too large and unwieldy to reach harmonious results in matter [illegible] special difficulty or much contention.

SPECIFICATIONS FOR STEEL [illegible]LS.

The report on steel-rail specifications [illegible] one of the principal subjects considered by [illegible] section. At last year's meeting [Eng. N[illegible] July 6, 1905, pp. 21-23, and July 13, pp. 27-[illegible] complete specification was submitted, and [illegible]le its general form received tacit approval [illegible] developed serious disagreement on sever[illegible]tails, three of which are vital points: (1) ma[illegible] drop test of rails from every fifth blow o[illegible]t [as against every blow]; (2) allowing 6 [illegible] maximum shrinkage after sawing [5 ins., t[illegible]re a lower temperature in the last rolling]; [illegible] shearing off and wasting the top of each ing[illegible]sufficient amount [as against a fixed per[illegible]ge of discard, or else such a discard that th[illegible] remaining ingot is solid metal].

It was decided last year to refer back [illegible] specifications to the committee, for further consideration of these points, and also for further effort to bring the specifications into [illegible]ement with those of the American Railway Engineering & Maintenance of Way Society and of the American Society of Civil Engineers. All the points in discussion were points of contention between rail manufacturers and railway engineers of test and maintenance, and the specifications represented a compromise. The rehearing during the past year had the object of securing more rigid requirements in the various points. The opposition of the manufacturers, who are fully represented in the committee, proved too strong, however, and the committee decided to stand pat on last year's specifications with but minor changes.

In discussion on the report, several strong objections were made to various of the clauses in contention, and then Mr. James Christie (Wissahickon, Pa.) expressed the opinion that since the interests involved in the rail specifications are very large, and since better specifications have been issued by the American Society of Civil Engineers and by the American Railway Engineering and Maintenance-of-Way Association, the whole subject should be referred back to the committee. A motion was therefore made by Mr. Robert Job (P. & R. Ry.) that the committee be instructed to prepare and submit at the next meeting a steel-rail specification "which will correct as far as possible the defective qualities in rails secured under existing specifications. After some discussion, in which some members urged that it was important to get out

a specification as soon as possible, while others urged that the society should proceed slowly so as to produce the best possible specification, the motion was carried. Mr. W. R. Webster (Philadelphia, Pa.), the chairman of Committee A, thereupon asked for instructions to the committee stating the important requirements which the specifications should include; inasmuch as the matter has been practically in its present status for two years and the committee has completed the best specification upon which it can agree, further labor will be useless unless the committee be instructed definitely as to the contentious features. Secretary Edgar Marburg (Philadelphia, Pa.), moved, however, that the chairman of Committee A be instructed to address to the Executive Committee his recommendations for facilitating the work of his committee, and this motion was carried.

Thus, the entire specification, which the committee had completed after several years' work, is rejected by the society, and the work is to be done over again de novo by the same committee.

SHIP MATERIAL.

The discussion of the subject of Specifications for Ship Material also developed much interest and some unnecessary excitement. The Executive Committee desired to bring before the meeting the question of whether it is desirable to draw up a specification for steel hull material, or, rather, revise the existing specification of the society; Mr. D. P. Stratton (American Bureau of Shipping, New York, N. Y.), and Mr. W. R. Webster had been delegated to open discussion of the subject, which they did in brief addresses. Mr. Stratton stated that a good specification drawn up by the American Bureau of Shipping is in existence and asked that the society approve this specification and adopt it as standard. He argued that foreign shipping bureaus are very antagonistic to American work and attempt to force their own specifications on American shipbuilders. Mr. Webster cited the strength requirements of Lloyd's specifications for ship material and suggested that these, with the chemical requirements of the bridge-material specifications of the American Society for Testing Material, would make a good specification for ship steel; Lloyd's does not specify any chemical composition for ship steel, which is out of line with the society's practice. Mr. W. A. Bostwick (Carnegie Steel Co., Pittsburg, Pa.), gave support to Mr. Stratton's proposals, stating that if the society prepares a hull-material specification it should base on American standards, that Lloyd's specifications represents essentially European practice, and that the American Bureau of Shipping is the American register. Mr. Stratton followed with a jingo speech on the strain "America for Americans." The meeting voted, however, on motion of Mr. Bostwick, that the entire subject of ship-material specifications be referred to the Executive Committee for consideration and reference.

SPECIFICATIONS FOR CAST-IRON AND FINISHED CASTINGS.

The report of Committee B, on Standard Specifications for Cast-Iron and Finished Castings, was submitted by Mr. Walter Wood (R. D. Wood Co., Philadelphia, Pa.). The committee has conferred with a committee of the American Water-Works Association which is engaged in formulating standard specifications for cast-iron water pipe. In the other subjects belonging to the work of Committee B, no changes are recommended in any of the adopted specifications. Mr Wood moved that Committee B be authorized to confer and collaborate with committees of the American Water-Works Association and other interests in preparing a harmonized standard specification for cast-iron water pipe; the motion was carried.

SPECIFICATIONS FOR STAYBOLT IRON.

Committee M, on Standard Specifications for Staybolts, reported that it was not prepared to present for adoption the tentative specifications submitted at last year's meeting. Several points still require further investigation before the committee feels warranted in presenting the specifications as completed. Concerning the method of piling iron for making the bar, however, satisfactory results can be reported; several manufacturers adopted this method during the past year, and the Baldwin Locomotive Works have used it for several years, with excellent results. Mr. M. H. Wickhorst (C., B. & Q. Ry., Aurora, Ill.), remarked that there is some doubt as to whether the committee was intended to work on staybolt iron or on staybolts; the committee has chosen the latter, but it seems to him doubtful whether it is desirable to go farther than staybolt iron. On his motion, the meeting voted to refer to the Executive Committee the question whether staybolt iron or staybolts shall be the subject of the committee's specifications.

SPEED OF PULLING IN TENSILE TESTS.

A lengthy report was made by Committee O, on Uniform Speed in Commercial Testing, Mr. P. Kreuzpointner (P. R. R., Altoona, Pa.), chairman. The committee conducted a series of tests, during the past year, on bar steels of 0.15 to 0.60% carbon, and on boiler steel, axle steel and tire steel. Over 400 test-pieces were prepared and sent to a large number of testing laboratories, at technical colleges and in industrial works, for testing in tension at prescribed speeds of pulling. The minimum and maximum speeds used were 1-64-in. and 6 ins. per min. It found, as the result of all these tests, that ultimate strength and elongation are little affected by increasing the testing speed. While there is generally a slight rising tendency, it is only slight, and would require a very much larger series of tests to portray it accurately. There is a greater rising tendency in the axle and tire steels, and at the same time there is less regularity of results. The committee recommends that speeds of 6 ins. per min. for boiler and bar specimens (8-in. testing length), and 3 ins. per min. for forging specimens (2-in. testing length), be considered maxima; below these limiting speeds it is safe to consider that speed has no influence on ultimate strength and elongation. The matter of elastic limit (or yield-point, the two terms being used as synonymous for the purposes of this report), is a much more delicate one, and the only conclusion warranted by the results is that the determination of elastic limit should be made at a slow speed, say ½-in. to ¾-in. per min. The determination of yield point at speeds materially higher is difficult and unsatisfactory, and finally becomes impossible, whether done by dividers or by drop-of-beam. In some cases, at high speeds, the beam did not drop at the yield-point, but merely showed some oscillations.

Mr. J. P. Snow said, in discussion, that since the usual methods of testing do not determine the true elastic limit (but merely the yield-point), and since the speeds recommended for determining the latter are too slow for practice, it would be better to eliminate from the society's specifications the requirement that the elastic limit must be recorded. Prof. E. Marburg, however, argued that the proper way is to run on slow speed until the yield-point is reached, and then finish the test on high speed; in this way the yield-point would be reliably determined and yet the time-saving of the high speed secured, since the range of motion up to the yield-point is only a very small part of the total elongation. There was some discussion on the terms elastic limit and yield-point, which led Mr. J. A. Kinkead (American Locomotive Co., Schenectady, N. Y.), to remark that in "commercial" testing these terms are always identical, and that in rough specimens the cracking of the scale gives a good check on the drop of the beam, in indicating the yield-point. Prof. M. Merriman (Bethlehem, Pa.) asserted that it would be a dangerous step to drop the determination of elastic limit from steel specifications, and argued that if present testing machines and methods do not give the elastic limit, then autographic methods must be used. Mr. W. C. DuComb (Riehle Bros. Testing Mach. Co., Philadelphia, Pa.) replied that the autographic method is too slow to be applied to all test-pieces in commercial work. Even the use of dividers for determining the yield-point is too much to ask of manufacturers in their mill laboratories. He also suggested that it would aid builders of testing machines if the committee drew up a standard set of speeds for testing machines, and moved that it be instructed to do so; the motion was referred to the Executive Committee for consideration.

It developed during the discussion that the committee has not as yet done any work on iron, but this material will be taken up in the coming year. The chairman expressed the opinion, based on long experience in the Altoona laboratory, that no appreciable influence of speed of testing is to be expected in the case of iron.

HEAT TREATMENT OF STEEL.

Committee F, on Heat Treatment of Iron and Steel, is headed by Prof. H. M. Howe. The report of the committee consisted of the description, by Prof. Howe, of a new furnace designed and built for the purpose of heating test-specimens for the investigations of the committee. The furnace contains two clay muffles, one within the other, placed in a lined gas-furnace. The gas-flame circulates around the outer muffle and heats it more or less uniformly; the muffle heats the inner muffle wholly by radiation, hence with great uniformity, and this latter similarly heats by pure radiation the specimen of steel, set vertically inside the inner muffle. A furnace of similar nature, but of more compact construction (and arranged horizontally instead of vertically) was used by Mr. William Campbell (New York, N. Y.) in carrying out a set of heat-treatment experiments under a grant from the Carnegie Institution, which he described in a paper associated with the committee report. These experiments were made on six grades of 5-16-in. bars, of various carbon percentages (from 0.70% to 2.04%). The principal critical point for all specimens lay between 730° and 740° C. (or 710 to 700° when cooling). Eight test temperatures were selected, 650° C. the lowest and 1200° C. the highest. A specimen was placed in the furnace, brought up to required temperature, which took about 1 hr., then held at that temperature for 5-10 mins., then cooled gradually in the furnace. Microphotographs of the treated specimens were taken, and physical tests were made. The temperature of 760° C. gave the best results. The microphotographs showed that at higher temperatures the cementite aggregates into larger masses, and at 1070° C. the two highest steels had their cementite break down to graphite. The highest temperatures produced an exceedingly coarse structure. Special series of tests will be run, in the next year, to investigate more fully the variations at 760°, and in the range 1070°-1200° C.

Mr. James Christie remarked that, in present-day practice, bridge eye-bars are heated up to 1500, 1600 and even 1700° F. (950° C.) in the annealing furnace, while as late as seven years ago temperatures of 1300 to 1400° F. were the highest used. Mr. W. A. Bostwick claimed that there is a distinct time-element in heat treatment, quite apart from any mere lag of temperature. Generally in annealing the temperature must be run much higher than the theoretical point, by an amount depending on the mass of the piece. Mr. E. O. Acker (Bethlehem Steel Co., So. Bethlehem, Pa.) confirmed this, stating that the results obtained in tests on small pieces do not seem to apply directly to large pieces, and that large pieces do not benefit so much as small ones by heat treatment.

Prof. H. M. Howe suggested an explanation for this: A large piece naturally cools more slowly than a small one, and correspondingly its structure will be made up of larger crystals; the segregation along the crystal axes, which is one of the things corrected by annealing, requires longer heating in the case of the large crystals because the ingredients must diffuse a longer average distance. But even aside from the influence of size of piece, there are many puzzling results; for example a small piece, if heated and cooled very slowly, may show no improvement even though the heating passed beyond the improving temperature. This may have some relation to the results of treating large pieces, since their interior will generally heat and cool very slowly. The

question was raised, in the course of the discussion, whether the change of cementite in high-carbon steel to graphite at 1070° C., could be reversed and the graphite reconverted to cementite. Mr. Campbell replied that it can be done in steels of 1.60 to 1.70 carbon, where only a fine scattered graphitic dust is produced, but so far he has been unable to reconvert very high carbon steels. Mr. R. Moldenke (Watchung, N. J.), noted that in malleable castings a similar effect appears; when the temperature in the annealing furnace goes too high the charge is ruined and the iron becomes white. Mr. J. A. Beckett (W. A. Wood Mower & Reaper Co., Hoosick Falls, N. Y.), added that too rapid cooling of malleable castings also may make them white, which was confirmed by Mr. Moldenke.

STEEL SPRINGS.

Committee T, on the Tempering and Testing of Steel Springs, reported a bare beginning of work. Three sub-committees have been formed for dealing with separate phases of the subject. A form of testing machine and methods of testing will be recommended by the committee; the rigid type of testing machine is preferred.

Mr. J. A. Kinkead, chairman of the committee, presented an interesting set of load-deformation curves of semi-elliptic locomotive springs. The curves exhibit very clearly the influence of friction between the leaves of the spring. It was found that hammering the spring while under load causes the curves of increasing and decreasing load to approach each other, but if the hammering is extended beyond the end support, the friction is brought back to the amount of the ascending curve.

SPECIFICATIONS FOR FOUNDRY COKE.

Committee J, on Standard Specifications for Foundry Coke, submitted a report in which it recommended that the word "foundry" be eliminated from the name of the committee, so as to extend its scope over coke for all uses, and bring a larger number of coke users and testers into direct relation with its work. Mr. R. Moldenke, secretary of the committee, spoke of the changes which the coke industry has experienced in recent years, which changes are still continuing. The reduction of time of burning, without deterioration of product, and the advent of by-product coke, make it impossible to prescribe the same acceptance requirements to-day as were satisfactory some years ago. It is considered impossible, therefore, to present a satisfactory coke specification at the present time.

Mr. J. A. Holmes (Chapel Hill, N. C.) made some remarks, in connection with the report, on the results of the fuel testing work of the Geological Survey, done at St. Louis, Mo. Although the work up to the present has covered only a small number of the coals of the country, it has been found that many coals hitherto considered non-coking give a good coke. The by-product coking process, moreover, enables coking with many coals not suited to the bee-hive process, and with the addition of foreign substances and proper mixture of different coals it will be possible to coke many others. Within a few days past, Congress has appropriated the sum of $250,000 for continuing the fuel-testing work, and the results of the work will be very important.

Besides considering committee reports, the section occupied itself with the discussion of a number of papers on matters affecting iron and steel materials.

CORROSION OF IRON AND STEEL.

The subject of corrosion was opened by a short paper on "The Corrosion of Steel as Affected by Its Chemical Composition," by Mr. J. P. Snow (B. & M. Ry., Boston, Mass.). The author takes it as proved that ingot iron under most conditions corrodes faster than puddled iron. His own experience proves it clearly. In signal bridges, for instance, old bridges made of iron are still in good condition, while newer bridges of steel, located over the same tracks and only a few hundred feet away, are very badly eaten; in the latter, moreover, certain members which show much less corrosion than the others are on examination found to be iron—loop-end rods, for example. These remarks apply primarily to those parts of structures which are directly exposed to the blast of gases from locomotives; the other parts of signal bridges corrode very much less, and the differences between iron and steel are not nearly so pronounced. Many other examples from American and European experience could be cited in proof. Up to the present nothing has been learned concerning the cause or causes of the greater corrosion of steel. Greater or less protection by scale is not a factor in the matter, certainly not a factor of any importance. Chemical constitution must be looked to as a reason. There is some evidence in support of the view that the manganese in the metal is responsible. The subject merits investigation by a special committee of the society.

Mr. M. Toch (New York, N. Y.) read a paper on the subject "Electrolytic Corrosion of Iron and Steel," in which he reported experiments made to determine the well-known fact that iron and steel are corroded by electrolytic action. He finds that the anode, which alone corrodes, exhibits a covering of greenish scum produced by electrolysis, which scum is ferrous hydroxide; this breaks down to a dark mixture of oxides, which he considers a criterion of electrolytic action, as also the laminated structure of the rust. The old artifice of protecting a structure which is subject to electrolytic corrosion by burying a plate or block of zinc near it, and bonding the structure to the zinc, was recommended by the author.

In a paper entitled "The Relative Corrosion of Wrought-Iron and Steel," Prof. Henry M. Howe (New York, N. Y.) analyzed certain differences between service results and laboratory test results on corrosion. The argument of the paper has general bearing on "accelerated testing," and we therefore reprint the paper in full elsewhere in this issue. Professor Howe further remarked, on Mr. Toch's paper, that tests on spring or knife steel may greatly mislead, as the differences in corrosion between boiler and knife steel are much greater than the difference between boiler steel and wrought iron. The protecting effect of the zinc block method he fears would be very local only.

Mr. A. S. Cushman (Washington, D. C.), speaking on the same subject, remarked that he long ago observed that two sewing needles from the same pack will, if plunged into an electrolyte and connected through a galvanometer, show a flow of current. If two needles thus cannot be made near enough alike to be neutral to each other, it is inevitable that differences of electrolytic potential must occur in wetted iron or steel structures. Investigation of the differences of corrosion in steels of different grades and in a single piece of steel are needed. Mr. James Christie reported that a sheet-iron roof 20 to 30 years old, which he has in daily view, is still in good condition, though it has no protection except occasional painting. Other (steel) roofs nearby, of later date, have been renewed two or three times in a dozen years, though they are under the same conditions and as well cared for as the older roof. Formerly sheet-iron was as often used "black" as galvanized or tinned, but nowadays it is out of the question to use black sheets, on account of their very rapid rusting. Some of the differences between iron and steel sheets may be due to the fact that the puddled iron has a rougher surface and the coating—paint, tin, or spelter—takes hold better. But this argument cannot apply to cut nails, which nowadays are often found rusted in the kegs, an occurrence never noticed with iron nails. It seems to him very likely that the cinder in wrought-iron has a strong protective effect. Thus, wrought-iron welds better when not highly refined, that is when less slag has been squeezed out by working; the larger amount of slag probably protects the welding surface from oxidation, for which purpose a flux must be used with the more highly refined iron, which is more "dry." Mr. Christie suggested that it might be possible that a material will ultimately be discovered which, mixed with the steel, will reduce corrosion without greatly impairing its other properties.

Mr. F. N. Speller (National Tube Co., Pittsburg, Pa.) said that a long series of tests had been made by the National Tube Co. on the relative corrosion of wrought-iron and steel pipes. The laboratory tests showed the two metals to be about alike, the corrosion of steel being never more than 10% higher than of iron. Two locomotives on the company's railway were fitted at the same time one with new tubes, one with steel, the other with iron (knobbled); the engines were kept track of very carefully until the two sets of tubes were worn out; the results showed no difference between the two. In pipe-lines, sometimes a few lengths of wrought-iron pipe get into a steel line by chance, and in such cases the wrought-iron lengths exhibit much pitting as the steel. [Generally, the pitting of steel, as against the more uniform corrosion of wrought-iron, is quoted as a pronounced point of inferiority of steel.] Where a steel pipe failed by corrosion, the company has always taken special pains to obtain the coupling (pipe-couplings even to-day being generally of wrought-iron) in order to compare its corrosion with that of the pipe; the results are about the same. A new variety of steel recently produced by the company, suitable for making welded tubes, shows excellent resistance to corrosion. Prof. H. M. Howe quoted tests recently made by Prof. Bradley Stoughton of the steel in question; the corrosion of this steel was .0570 as against .0660 for a parallel test with puddled wrought-iron. Mr. J. J. Shuman (Jones & Laughlin, Pittsburg, Pa.) stated that an iron oil-well casing recently drawn after 7 years' service was found in perfect condition; steel casings in the same field, no older, are badly corroded. He remarked also that steel tubing, in storage piles, may have large, clean blue patches, but will be badly corroded over the rest of the surface; iron tubes generally exhibit uniform rusting. Mr. E. S. Cook (Warwick Iron & Steel Co., Pottstown, Pa.) reported that a corrugated iron roof laid in 1880 is in better condition to-day than a neighboring roof 14 years old, which was sold as iron, but really is steel.

Numerous other contributions to the subject were made, but we note only two further observations. Mr. Cushman stated that contrary to expectations it is the upper wires in woven steel wire fences which rust, while the wires near the ground remain bright for a long time. Mr. Speller remarked that the electrolytic potential difference between iron and its cinder is as high as 0.2 to 0.3 volt, which is greater than the voltage of the iron against any one of the elements which it contains.

The meeting thereupon voted that a committee be appointed for investigating the subject of Corrosion of Iron and Steel.

FIREBOX STEEL.

A paper by Mr. M. H. Wickhorst (C. B. & Q. Ry., Aurora, Ill.), entitled "Firebox Steel, Failures and Specifications," presented an excellent study of some of the operating factors which decrease the life of a firebox. While the study is very interesting in relation to the life of fireboxes, the interest to the society lay in the leading conclusions concerning the material: First, that firebox failures are due mainly to operating methods, the quality of the steel being not responsible; second, that the tendency of recent years to specify tensile strength limits of 55,000-65,000 lbs. for firebox steel secures a harder steel than the severe straining of fireboxes makes desirable, and that the older limits having a minimum between 50,000 and 52,000, and a maximum limit between 58,000 and 62,000 lbs. per sq. in., are preferable.

Mr. W. A. Bostwick asked whether the author considered that firebox steels with .035 and .020% sulphur, respectively, showed any difference in service; the author replied that no trouble from sulphur had been noticed, and that .035 should be amply low enough.

SEGREGATION IN PLATE INGOTS AND VARIATIONS IN TENSILE STRENGTH OF STEEL PLATES.

An interesting paper by Mr. C. L. Huston (Lukens Iron & Steel Co., Coatesville, Pa.)

throw much light on the strength conditions of universal-mill steel plates rolled directly from the ingot. The plates (mild steel) all show lower tensile strength around the border, while the center is quite uniform and from 5,000 to 8,000 lbs. stronger. This is due to segregation of carbon; the carbon content and the tensile strength vary together. The facts are important because the specimens are cut from the edges and ends of plates, and will thus consist of the softest part of the plate; if a strength of 60,000 lbs. is required on test, the body of the plate will thus be considerably higher (harder). The effect is more serious in thick plates, where, because of less hardening due to rolling, a higher-carbon material must be used anyway to secure the same test strength as in a thinner plate. We hope to reprint this paper fully in another issue.

In reply to a query the author stated that the small holes which lie in a sheet a short distance below the surface of the ingot all round, as shown in sections of ingots exhibited, do not remain as seams in the rolled plate, but weld together in rolling, except where an unusually large blowhole occurs. The absence of such seams is shown by tensile tests on transverse strips. However, the ingot requires to be rolled quickly at a good heat, to accomplish this closing.

The author's remarks having favored somewhat lower tensile-strength limits for plate steel than are now used, Mr. J. P. Snow supported this advocacy of lower carbon, and asked for a vote of the meeting on the desirability of raising the specified tensile strength (desired) from 60,000 to 62,000, as had been proposed during the past year. The lower tensile strength is especially desirable because of the tendency toward thicker metal, thicker angles, etc., in bridge work. He held, also, that the same reasons would affect ship material, and that the specification in project should not set higher limits than for bridge steel. Mr. W. A. Bostwick also favored lower tensile strength, or at least reaffirming the present limits. It was ruled, however, that in default of contrary motion no such vote was necessary.

In connection with the matter of segregation, Prof. H. M. Howe, at another meeting, advocated that in testing steel for chemical composition, check drillings should be taken from the part most subject to segregation; and that specifications should, besides fixing the limits of composition for the ladle test, specify maximum limits of increase in these special segregation drillings. This applies mainly to determinations of carbon, phosphorus and sulphur. Mr. W. R. Webster remarked that something of this sort is done in the specifications for steel axles, which define a point at a given distance from the center as the location for check drillings, and fix maximum amounts of increase from ladle test to these drillings.

Two other papers were presented before the section, one by title and the other in full, with little or no discussion. The first is "The Beneficial Effect of adding High-Grade Ferro-Silicon to Cast Iron," by Mr. A. E. Outerbridge, Jr. (Wm. Sellers & Co., Philadelphia, Pa.). It presents a method for simultaneously softening and strengthening cast-iron in the ladle, by the addition of 50% silicon-iron alloy, in coarse powdered form, sprinkled into the ladles full of iron.

The other paper, "Tension Tests of Steel Angles," by Prof. F. P. McKibben (Boston, Mass.), described tension tests made on angles riveted to gusset-plates in various ways; they afford a comparison between different methods of attachment, and between the respective strengths of the riveted angle and of a regular tension test-piece cut from the angle. This paper will be reprinted in our next issue.

THE HUDSON-FULTON CENTENNIAL is to be celebrated by New York City and a report has just been made by a committee on the form which the celebration is to take. It is proposed to celebrate at one time the fourth centennial of Hudson's discovery and the first centennial of Fulton's trial of the "Clermont." It is proposed that the celebration occur during the week beginning Sept. 20, 1909. The following is extracted from the committee's report:

As both of the events to be commemorated were of a nautical character, it seems appropriate, first of all, that there should be a demonstration in the form of a naval festival upon the river. In this pageant we recommend that the navies of the United States and foreign nations, particularly the Netherlands and Great Britain, be invited to participate; also the American merchant marine, excursion boats and yacht clubs contiguous to New York Harbor, and the Hudson River. We recommend that this naval procession include fac-simile reproductions of the "Half Moon" and the "Clermont," the former to be equipped with an auxiliary motor. We recommend that the government of the Netherlands be invited to send the model of the "Half Moon" and that American steamboat interests be invited to contribute the model of the "Clermont."

Upon the second day we recommend that there be a land parade and literary exercises.

We recommend that a third day be devoted to the dedication of memorials. The Hudson Memorial Bridge, extending from Inwood Hill to Spuyten Duyvil Hill, across Spuyten Duyvil Creek at its confluence with the Hudson River, is assured by action already taken by the municipal authorities. We recommend that this commission formally adopt the bridge as part of the commemoration. We also recommend that the viaduct projected by the city to cross Dyckman St., to connect the southern end of Inwood Hill with Washington Heights on the south, be called the Fulton Memorial Viaduct, and, if possible, dedicated at the same time.

We recommend that about 75 acres of the northern portion of Inwood Hill be taken for a public park. This park, which has already been recommended by influential civic societies and by the city's own engineers, would secure the last portion of Manhattan Island remaining in almost its primeval condition. From this beautiful wooded knoll, appearing to us to-day almost as it did to Henry Hudson nearly 300 years ago, a more extended view up Hudson's river can be obtained than from any other part of the island.

A SERIES OF TESTS were recently made on one of the four 5,000-KW. Curtis steam turbines, which have been in commercial service for about one year in the Fisk Street Station of the Commonwealth Electric Co., Chicago, Ill.

The unit tested was a 5,000-KW. turbine, direct-connected to a 6-pole, three-phase, 5,000-KW., 9,000-volt, 500 r. p. m., revolving field generator. All the tests, except those for speed, were made under regular commercial load conditions. In making the speed test the current was absorbed by a water rheostat, composed of plates placed in the Chicago River, in order to obtain a constant load.

The load was measured by special indicating wattmeters, connected to independent potential and current transformers, the load readings being recorded every two minutes. All tests were made at about 150° F. superheat, the temperature being read by calibrated mercury thermometers, temperatures and pressures being recorded every five minutes.

The results of the test, after being reduced to 150° F. superheat, 1½ ins. mercury back pressure and 175 lbs. (gage) steam pressure, are summarized as follows:

R. P. M. = 500.								
¼ Load	= 2,500 KW.,	Water Rate	= 17.74 lbs.	per	KW.	hr.		
¾ "	= 3,750 "	"	= 17.08 "	"	"	"		
Full "	= 5,000 "	"	= 16.62 "	"	"	"		
1¼ "	= 6,250 "	"	= 16.52 "	"	"	"		
1½ "	= 7,500 "	"	= 16.90 "	"	"	"		
R. P. M. = 650.								
¾ Load	= 3,750 KW.,	Water Rate	= 16.35 lbs.	per	KW.	hr.		
Full "	= 5,000 "	"	= 16.07 "	"	"	"		
1¼ "	= 6,250 "	"	= 15.88 "	"	"	"		
1½ "	= 7,500 "	"	= 15.80 "	"	"	"		

A RUNAWAY CAR on the line of the Orange Mountain Traction Co., at Orange, N. J., on June 24, caused the death of one man and the injury of twelve others. All of those injured jumped from the car as it rushed down the mountain, the one man who remained on the car escaping with slight bruises.

The line was formerly a cable incline road running in a straight line up Orange Mountain, a distance of a little over a mile, the grade ranging from 15 to 20%. The cable road was a financial failure, and a company had recently undertaken to transform the road to an electric line. The road had not yet been opened to the public, and this trip was undertaken to test the emergency brakes.

It is stated that an auxiliary safety cable was to be eventually used, but it was not attached to the car when the accident occurred, and the sole reliance for holding the car on the grade was a patent emergency brake said to be invented by Patrick Flood, of Albany, N. Y. As described in the published reports, the brake consisted of a metal wedge designed to chock the wheels on the downhill side.

The party on the car consisted of members of the construction gang, Mr. Flood and his assistants, and a representative of the Public Service Corporation. The car started up the grade making frequent stops to test the brakes, which worked satisfactorily until a deep cut at the summit was reached, where the steepest grade of the line occurs. Here the emergency brake slipped with a series of jumps, and in a few seconds the car was going down the hill at high speed, and the efforts of those on board to control it were fruitless. The accident is a notable illustration of the results of ignorance in handling a very simple engineering problem.

A COMPRESSED AIR BLOW-OUT in the southernmost of the four tubes of the Pennsylvania R. R. tunnels across the East River at New York City on June 20 resulted in the death of two men by drowning and a couple days' delay in operations. The shield was working about 500 ft. out from the shore shaft on the Manhattan side in soft material, sand with thin layers of clay, when the blow-out occurred with an inrush of water at about mid height of the face. Two men who were working in front of the shield diaphragm were caught and drowned, but all the others escaped through the bulkhead air lock uninjured except from bruises caused by the struggle to escape.

A WATER-PROOFING COMPOUND for cement and concrete, invented by Mr. Spencer B. Newberry, is being placed on the market by the Sandusky Portland Cement Co., of Sandusky, Ohio. The compound is a dry powder which is mixed with the cement in the proportion of 4 to 8 lbs. to the barrel. It does not affect the strength or color of the concrete and tests show a reduction in the absorption to only a sixth of the amount of water absorbed by ordinary concrete. In a 24-hour test the percentage of water absorbed was only 1.27% in a sample made with 8 lbs. water-proofing compound to the barrel. In other words, concrete thus water-proofed appears to absorb little more water than a good paving brick. The compound is being used by concrete block makers and promises also to find a wide field in the water-proofing of cellar walls.

A TYPHOID EPIDEMIC at Ellinwood, Burton County, Kans., has been investigated by Dr. S. J. Crumbine, Secretary of the State Board of Health. The following is an excerpt from his report:

"Upon arrival in this city we found a town of about 1,000 inhabitants, scattered over an area of one-half square mile. The location is in the Arkansas valley, but a short distance from the Arkansas River, the surrounding country almost as level as a floor, thereby making the drainage very imperfect. The soil is composed of a sandy loam. The water supply of the city is gotten from wells, in most instances driven into the soil 20 to 25 ft. deep, where the first water layer is encountered, from which almost all of the residents of the city are supplied.

"A number of modern houses have constructed cesspools for the disposal of sewage. Each one of these cesspools was discovered to have been dug to the water-bearing stratum and the contents of the cesspool allowed to drain off through this sand and gravel. Three of the cesspools are located immediately west of the principal point of infection.

"The conclusions drawn, therefore, from the location of these cesspools, the location of the principal source of infection, the location of most of the cases of typhoid, would seem to indicate the necessity of the abandonment of all cesspools in this city immediately, filling them up with earth to above the water stratum; then, if desired to be used as cesspools, to require them to be cemented water-tight as a cistern and be emptied as occasion required.

"It is suggested that these cesspools should be cemented water-tight, and that drainage therefrom might be had by four openings near the top, just below the frost line, through perforated tile drains running in various directions. Such a constructed cesspool would act somewhat in the capacity of a septic tank, liquefying a great deal of its contents by bacterial action, and supplying trees and growing vegetation with subirrigation. The top should be so constructed that the sludge could be pumped out when occasion required. Such a method would be free from contaminating the source of water supply, and would still permit the construction of modern homes in towns too small to have sewerage systems."

It will doubtless occur to many of our readers that the proposed cementing of a cesspool to make it water-tight, so that no pollution may reach the ground water below is a task practically impossible to accomplish. What should be done is to introduce a pure water supply, drawn from sources entirely outside the polluted area. Wells in any closely built-up village are apt to be a source of disease; but wherever cesspools are introduced, there can be no such thing as a safe ground water supply in their vicinity.

THE COAL MINED IN ALABAMA during the year 1905 amounted to 11,860,069 short tons, its value at the mines being $14,387,721. This was an increase over 1904 of 5.3% in tonnage and 6.7% in value, while, when compared with 1903, the year of largest production heretofore, it shows an increase of 211,745 tons in quantity and $140,923 in value.

THE PRODUCTION OF PORTLAND CEMENT in 1905, according to the United States Geological Survey, amounted to 35,246,812 bbls. valued at $33,245,867. This is an increase over the output of 1904 of 8,740,931 bbls. and $9,890,748.

THE VALUE OF BUILDINGS erected in 47 of the leading cities of the United States during the year 1905 was, according to the United States Geological Survey, $640,555,641, which is a considerable increase over the previous year, amounting to $173,855,931. The largest expenditure was made in New York City, which put up buildings to the value of $178,032,527; Brooklyn, N. Y., comes next with an expenditure of $73,017,701; Chicago, Ill., is third with $35,000,000, and Philadelphia, Pa., fourth with $34,416,745.

A STUDY OF WISCONSIN ROADS is being conducted by the Geological and Natural History Survey of the State of Wisconsin. With the ultimate object of spreading knowledge of how to build and repair roads effectively and economically, the Survey is now engaged in collecting comprehensive information on the condition of roads and the methods used in building and repairing roads in the state. Particular attention is being paid to the common dirt roads; these will make up by far the largest part of the road mileage for many years to come, and therefore information as to the cheapest and best methods of making such roads will be of greatest value. As one step in its study, the Survey is sending to each town clerk in the state a letter enclosing a list of questions concerning the roads in his township; a complete record of road conditions in the state will be compiled by this means. The investigation of roads is in the hands of W. O. Hotchkiss, Geologist in charge of Economic Geology, Madison, Wis., who will doubtless be glad to receive information or answer questions bearing on the work.

PERSONALS.

Mr. W. Irwin Kettlewell, Engineering Aid, has resigned from the Reclamation Service.

Mr. W. L. Darling, Chief Engineer of the Northern Pacific Ry., has been elected Vice-President of the Portland & Seattle Ry.

Mr. Byron Layton has been appointed Engineer of Maintenance of Way of the Chicago, Indiana & Southern R. R., with office at Hammond, Ind.

Mr. W. E. Bogart has been appointed Chief Engineer of the Wichita Valley Ry. to succeed Mr. J. W. Field, resigned. His headquarters are at Seymour, Tex.

Mr. Douglass P. Morrison has been appointed Electrical Engineer of the Pittsburg & Lake Erie R. R., with office at Pittsburg, Pa. He succeeds Mr. G. M. Campbell, resigned.

Mr. Joseph W. Ripley, M. Am. Soc. C. E., has been appointed General Superintendent of Locks in Panama. He will engage in the work of designing and constructing the locks for the canal.

Mr. E. D. Martinez, Civil Engineer and Head of the Construction Department of the Houston Electric Co., Houston, Tex., has resigned to enter the employ of the Street Railway Co. of Memphis, Tenn.

Mr. Paul A. Hartung has resigned as Resident Engineer of the Indianapolis Southern Ry., at Bloomfield, Ind., to become Locating Engineer on the Union Pacific R. R. His headquarters are at Omaha, Neb.

Mr. Gardiner C. Sims has resigned as General Manager of the Marine Engine & Machine Co., of Harrison, N. J., having recently been elected President of the William A. Harris Steam Engine Co., of Providence, R. I.

Mr. L. F. Loree, M. Am. Soc. C. E., formerly President of the Baltimore & Ohio R. R., has been elected a Director of the Kansas City Southern Ry. and will act as a technical expert in regard to the improvement of the property and methods of management.

Mr. George I. Rockwood, M. Am. Soc. M. E., a well-known engineer, has been appointed to the Professorship of Steam Engineering in the Worcester Polytechnic Institute. Mr. Rockwood is a graduate of the Worcester Polytechnic Institute and has consulting offices at Worcester, Mass.

Mr. E. D. Harshbarger, Assoc. M. Am. Soc. C. E., until recently First Assistant Engineer of the Bureau of Filtration, Department of Public Works, Pittsburg, Pa., has resigned and accepted the position of Engineer of the Pitt Construction Co., Inc., Engineers and Contractors, 821 Fulton Bldg., Pittsburg, Pa.

Mr. Harrison P. Eddy, Superintendent of Sewers for Worcester, Mass., has been retained as Consulting Engineer to a special Commission now planning a new sewerage system for the city of Louisville, Ky. The new system will cost about $4,000,000, and will include a trunk sewer about 10 miles long.

Mr. E. E. Haskell, M. Am. Soc. C. E., Principal Assistant Engineer of the United States Lake Survey, has been appointed Dean of the College of Civil Engineering at Cornell University, the office having been vacant since the death of Prof. E. A. Fuertes in 1904. Mr. Haskell is a graduate of Cornell and the inventor of the Haskell current meter, and has held important positions with the U. S. Coast and Geodetic Survey, the Mississippi River Commission, and the Corps of Engineers, U. S. Army.

Mr. E. W. Grice has been appointed General Superintendent of the West Virginia Grand Division of the Chesapeake & Ohio Ry., in charge of operation and maintenance, with office at Hinton, W. Va. Mr. J. R. Cary has been made Superintendent of the Hinton Division with office at Hinton, W. Va.; Mr. C. P. Snow, Superintendent of the Huntington Division with office at Huntington, W. Va., and Mr. J. W. Haynes, Superintendent of the Greenbrier Division with office at Ronceverte, W. Va.

Mr. N. B. Roper, formerly Chief Engineer of the Cananea Consolidated Copper Co., Cananea, Mex., has been engaged by the Power & Mining Machinery Co., of Milwaukee, Wis., as Mining Engineer, to represent them at their New York Sales Office, 52 William St. Mr. W. A. Lieblein has been made Manager of the Salt Lake City branch, with offices at 215 Commercial Club Bldg. Mr. Lieblein was formerly Manager of the Sale Lake City office of the Rand Drill Co. Mr. W. C. MacDowell, formerly connected with the New York City office, has been made Manager of the El Paso, Texas, office.

The transportation department of the Southern Ry. will shortly be divided into four districts, each of which will be in charge of an assistant general superintendent, as follows: First district: Washington, Danville, Richmond and Norfolk divisions; Mr. E. H. Coapman. Second district: Charlotte, Charleston, Savannah and Jacksonville divisions; Mr. J. N. Seale. Third district: Atlanta, Knoxville, Nashville and Asheville divisions; Mr. G. R. Loyall. Fourth district: Birmingham, Memphis and Mobile divisions; Mr. W. W. Foreacre. Mr. C. L. Harris will become Superintendent of the Birmingham division.

Mr. William C. Nixon, Vice-President and General Manager of the Gulf, Colorado and Santa Fe Ry., will become, on Aug. 1, Vice-President and General Manager of the Atchison, Topeka & Santa Fe Ry. Mr. Nixon is a native of Illinois and began his railway service in February, 1879, as a bridge carpenter, on the Burlington & Missouri River R. R., in Nebraska. He accepted a position as watchman with the Atchison, Topeka & Santa Fe in August, 1879, and has since been steadily promoted, having been made Superintendent of Terminals at Kansas City in March, 1889, Superintendent of the Chicago District the following April, Superintendent of Terminals at Chicago in February, 1891, General Agent at Chicago July 1, 1896, Superintendent of the Chicago Division Jan. 1, 1897, General Superintendent of the Gulf, Colorado & Santa Fe Ry. Jan. 1, 1900, and two years later General Manager of the Gulf Division of the Frisco System.

Obituary.

Lewis E. Gray, Superintendent of Water-Works in North Tonawanda, died at Buffalo, N. Y., June 22, aged 45 years.

Robert Bruce Burns, Chief Engineer of the Coast Lines of the Atchison, Topeka & Santa Fe Ry., died at Los Angeles, Cal., June 21, aged 54 years.

William A. Akers, a mining engineer of Los Angeles, Cal., was killed June 16 in an automobile collision in that city. Mr. Akers was about 40 years old and formerly lived at El Paso, Texas.

William Cudworth, at one time Chief Engineer of the Northeastern Ry., of England, died June 4 at the age of 91. He was Chief Engineer of the historical Stockton & Darlington Ry., and when that was consolidated with the Northeastern Ry. in 1873 he was appointed to the same position which he held until his retirement from active professional life.

B. H. Fisher, Chief Engineer of the North Shore R. R. of California, died at his home in Sausalito, Cal., May 21, aged 46 years. Mr. Fisher was a graduate of Cornell University and was well known on the Pacific Coast, having been connected with the Atchison, Topeka & Santa Fe Ry., Albuquerque Electric, Water and Gas Works, Oakland-Alameda Belt Ry., and the North Shore R. R.

Stanford White, member of the well-known firm of McKim, Mead & White, Architects, New York City, was shot and instantly killed by Harry Thaw, at the Madison Square Roof Garden, Monday night, June 25. Mr. White was born Nov. 9, 1853, in New York City, and after completing his studies in the public schools he received his architectural training in the offices of Chas. D. Gambrill and H. H. Richardson. In 1878 he sailed for Europe and spent two years in traveling and studying, returning to New York in 1880, when he entered into partnership with Chas. F. McKim and William R. Mead, under the firm name of McKim, Mead & White. He was the designer of Madison Square Garden, the Century and Metropolitan club-houses, some of the principal buildings of the University of New York and the University of Virginia. Mr. White was a member of the American Institute of Architects and held the honorary degree of A. M. from the University of the City of New York.

Francis William Webb, for many years Chief Mechanical Engineer of the London & Northwestern Ry., of England, died at Bournemouth, England, on June 4. He was born in 1835, and in 1851 entered the works of the London & Northwestern Ry., at Crewe, as a pupil of Mr. Francis Trevithick, the first Locomotive Superintendent of that railway. In 1859 he was made chief draftsman at Crewe, under the late Mr. Ramsbottom. In 1866 he left to become manager of the Bolton Iron & Steel Works, but on Mr. Ramsbottom's retirement in 1871 Mr. Webb accepted the position of Chief Mechanical Engineer of the railway. He is best known in connection with his three-cylinder compound locomotives, which, however, he ultimately abandoned in favor of a four-cylinder system. The railway shops at Crewe not only built locomotives, but included also a Bessemer steel plant, a rail rolling mill, presses for steel ties, and turned out quantities of material for track and signal work. He was a prolific inventor, and his inventions covered a wide range of machinery and appliances. Among these was a revolving mold for driving-wheel centers, the centrifugal action causing the molten metal to become very dense, thus preventing flaws or cavities. As early as 1873 he began the use of steel for boilers and fireboxes. He was elected an Associate Member of the British Institution of Civil Engineers in 1857 and a Member in 1872, while at one time he held the position of Vice-President. He resigned his position with the London & Northwestern Ry. in 1903, on account of ill-health.

ENGINEERING SOCIETIES.

COMING MEETINGS.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION.
June 29 to July 4. Annual meeting at Ithaca, N. Y. Secy., M. S. Ketchum, University of Colorado, Boulder, Colo.

AMERICAN WATER-WORKS ASSOCIATION.
July 10 to 14, inclusive. Annual meeting at Boston, Mass. Secy., J. M. Diven, 14 George St., Charleston, S. C.

AMERICAN SOCIETY OF HEATING AND VENTILATING ENGINEERS.
July 19 to 20. Semi-annual meeting at Chicago, Ill. Secy., W. M. Mackay, P. O. Box 1818, New York City.

AMERICAN INSTITUTE OF MINING ENGINEERS.
July 23 to Aug. 4. Joint meeting with the Iron and Steel Institute at London. Secy., R. W. Raymond, New York City.

CONCRETE BLOCK MACHINE MANUFACTURERS' ASSOCIATION.
Aug. 8, 9. Annual meeting at Detroit. Secy., S. L. Wiltse, Jackson, Mich.

INTERNATIONAL ASSOCIATION OF MUNICIPAL ELECTRICIANS.
Aug. 15 to 17. Annual convention at New Haven, Conn. Secy., Frank P. Foster, Corning, N. Y.

CONCRETE BLOCK MACHINE MANUFACTURERS' ASSOCIATION.—The second annual convention will be held at Detroit, Mich., on Aug. 8 and 9. A program has been arranged, and members are invited to express their views of the concrete block and cement brick industry. The Secretary is Mr. S. L. Wiltse, of Jackson, Mich.

THE ASSOCIATION FOR THE PROMOTION OF HYGIENE AND SALUBRITY IN DWELLINGS.—The second international congress of the association will be held at Geneva, Italy, Sept. 4 to 10. The membership is divided into two divisions, full and associate. Full members pay a subscription of 20 fr. ($4.00), which entitles them to all publications of the congress, to participation in all the fetes, excursions and receptions, to a reduction of railway fares on certain lines, etc. Associate members pay a subscription of 10 fr. ($2.00), which entitles them to all the privileges of full members, except the publications of the congress.

The object of the congress is stated as follows:

To study the health conditions under which buildings designed for habitation are constructed and established, to seek out the improvements of which they are susceptible in construction, management and maintenance, and to decide upon the practical means of obtaining, with the aid of town and city authorities, proprietors, architects, engineers and builders, as well as the inhabitants themselves, the application of hygienic principles.

The program of the congress is divided into general and sectional meetings. The subjects of the general meeting are: "The necessity and means of the transformation of unhealthy blocks of houses in cities," "The application of sanitary legislation in the case of unhealthy dwellings" and "Hygienic systems of heating." Sectional meetings are divided into eleven sections, viz.: "Houses in Cities (middle class)." (a) The housing of servants. (b) The danger of underground flats. (c) The housing of caretakers, cooks, etc. "Workmen's Dwellings." (a) Their distribution in cities. (b) The means of insuring their wholesomeness and salubrity. (c) The role of public authorities and private action in the matter. (d) Discussion of the question of gardens and plots of ground for workmen's cottages. "Rural Dwellings." (a) The depopulation of country districts. (b) The housing of farmhouse servants and laborers. (c) The importance of a speaking knowledge of the laws of health among the peasantry; "Furnished Lodgings and Hotels;" "Hospitals," sanitary conditions necessary in hospitals, asylums, and sanitoriums; "Military Establishments," the present types of barracks, the general conditions they should fulfill in relation to climate and country; "Public Halls and Industrial Establishments," references to the hygienic situation and construction of theaters, concert rooms, shops, factories, etc.; "School Houses," vitiation of air in public and private school rooms, and how to remedy it; "Means of Travel," special papers on railway carriages, trains and steamers; "How to ameliorate the sanitary conditions of old quarters in town or city, or of the monuments of the past, while at the same time preserving their picturesque and artistic character;" Legislation, Sanitary Offices, Statistics and Sanitary Certificates. Secy., M. Albert Wuarin, 1 Rue des Moulins, Geneva, Italy.